TE
662
.A3
no.
FHWA-
RD-
75-11
c.2

eport No. FHWA-RD-75-11

ENTATIVE PAVEMENT AND GEOMETRIC DESIGN CRITERIA FOR MINIMIZING HYDROPLANING

B. M. Gallaway and others

Dept. of Transportation
AUG 4 1976
Library

February 1975
Final Report

This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

NOTICE

This document is disseminated under the sponsorship of the U.S. Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof.

The contents of this report reflect the views of the Texas Transportation Institute which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the Department of Transportation.

This report does not constitute a standard, specification, or regulation.

FHWA DISTRIBUTION NOTICE

Sufficient copies of this report are being distributed by FHWA Bulletin to provide a minimum of two copies to each regional office, two copies to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

TE
662
.A3
no.
FHWA-
RD-
75-
11
C.2

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-75-11		

4. Title and Subtitle	5. Report Date
TENTATIVE PAVEMENT AND GEOMETRIC DESIGN CRITERIA FOR MINIMIZING HYDROPLANING	February 1975
	6. Performing Organization Code

7. Author(s)	8. Performing Organization Report No.
B. M. Gallaway, D. L. Ivey, H. E. Ross, Jr., W. B. Ledbetter, D. L. Woods and R. E. Schiller, Jr.	

9. Performing Organization Name and Address	10. Work Unit No.
Texas Transportation Institute Texas A&M University College Station, TX 77843	FCP 31H2-212
	11. Contract or Grant No. Contract DOT-FH-11-8269
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Federal Highway Administration Offices of Research and Development Washington, D. C. 20590 HRS-12	Final Report Phase I
	14. Sponsoring Agency Code HRS-12

Dept. of Transportation
AUG 4 1976
Library
S0495

15. Supplementary Notes

FHWA Contract Manager: Glenn G. Balmer (HRS-12)

16. Abstract A comprehensive literature review, multi-state questionnaires, mathematical modeling, computer simulations, field testing and data correlations were used to establish tentative criteria relating to geometric and pavement surface texture and cross slope to minimize highway hydroplaning.

More specifically the authors have covered in detail the empirical indications of hydroplaning as determined from interpretation of hard data developed with a spin down trailer in a hydroplaning trough, skid number trailers including drag link and torque and extensive tire and pavement testing under simulated rain. Precise measurements of surface drainage were examined and equations relating pavement texture and cross slope and rainfall density were developed. Random and patterned textures covering the complete range of real world pavements were included.

Vehicle control as related to variation in cross slope was simulated with the HVOSM. Travel with constant velocity along a tangent path and a lane change maneuver at 60 mph (97 km/h) involving the traversal of a crown were tested for driver demand and vehicle control. A determination of deficiencies in existing surface drainage design methodology for sag vertical curves was conducted by state surveys and representative sites were selected for further study.

A summary of tentative criteria to reduce hydroplaning is presented along with recommendations for construction of flexible and rigid pavements to minimize hydroplaning.

Recommendations are presented for Phase II of this study wherein it is expected that the tentative criteria established in Phase I will be further confirmed and the more critical gaps in the current information will be filled.

17. Key Words	18. Distribution Statement
Skid number, texture, available friction, hydroplaning, tire-pavement interaction, vehicle maneuvers, pavement surface design, criteria, cross slope, driver requirements, math model, computer simulation, flow-phenomena, rainfall intensity	No restriction. This document is available through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	208	

Form DOT F 1700.7 (8-69)

PREFACE

This is the first report issued in a two-phase study under DOT Contract FH-11-8269, which deals with "Pavement and Geometric Design Criteria for Minimizing Highway Hydroplaning".

The report presents tentative criteria developed by a seven-man team for pavement surface texture and cross slope and suggestions for modifications in pavement geometrics to minimize hydroplaning.

Recommendations are presented for work to be accomplished in Phase II of the study which efforts will fill gaps revealed by the research team that developed the Phase I criteria.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to Mr. Glenn G. Balmer, contact representative of the Federal Highway Administration, for his assistance in securing vital background information for the study and for his counsel and devoted interest in assuring the successful execution of the objectives of the study.

The secretarial and technical staff of the Safety Division is commended for the preparation of the manuscript.

TABLE OF CONTENTS

CHAPTER		PAGE
	GENERAL INTRODUCTION	1
I	THE PHENOMENON OF HYDROPLANING	3
II	EMPIRICAL INDICATIONS OF HYDROPLANING	16
	Interpretation of Data from Different Test Mechanisms	16
	Spin Down Trailer	16
	Skid Number Trailer - drag link	20
	Skid Number Trailer - torque	22
	Interpretation of Data from HPR Studies 147, 138 and 141	23
III	PAVEMENT DRAINAGE	45
IV	SURFACE TEXTURE, SKID RESISTANCE AND ACCIDENT FREQUENCY	63
	Random Textured Pavements	79
	Equations Used for Annex Data (Pads 1, 3, 4, 5, 6, 7, 8)	81
	Equations for Pad 2 Jennite Flush Seal TXD = 0.002 in. (0.051 mm)	86
	Equations for Pad 9 Painted Concrete TXD = 0.033 in. (0.838 mm)	86
	Equations for Pad 1 Portland Cement Concrete TXD = 0.036 in. (0.914 mm)	87
	Patterned Texture Pavements	88

TABLE OF CONTENTS (continued)

CHAPTER		PAGE
	East Bypass Equations	97
	Transverse Textured Pavements	97
	Longitudinally Textured Pavements	97
V	PAVEMENT CROSS SLOPE CRITERIA AS RELATED TO VEHICLE CONTROL	104
	Introduction	104
	Research Approach and Evaluation Criteria	104
	Parameters Investigated	106
	HVOSM	106
	Tangent Path Results	106
	Lane Change Results	112
	Conclusions	118
VI	A DETERMINATION OF DEFICIENCIES IN EXISTING SURFACE DRAINAGE DESIGN METHODOLOGY FOR SAG VERTICAL CURVES	120
	Introduction	120
	Design Elements Considered	120
	Examination of Design Policies	122
	Summary of Responses to Sag Vertical Curve Drainage Design Questionnaire	124
	Statement of Deficiencies	136
	Recommendations of Possible Study Sites	139
	Summary of Site Recommendations	140

TABLE OF CONTENTS (continued)

CHAPTER		PAGE
VII	SUMMARY OF TENTATIVE CRITERIA TO REDUCE HYDROPLANING	143
VIII	TENTATIVE RECOMMENDATIONS FOR CONSTRUCTION	158
	Dense Surfaces (Impermeable)	159
	Open Surfaces (Permeable)	159
	Construction of Asphaltic Concrete Surfaces	164
	Construction of Portland Cement Concrete Surfaces	168
APPENDICES		
A	ALIGNING TORQUES FOR TANGENT PATH	170
B	RESULTS OF LANE CHANGE MANEUVERS	178
REFERENCES		186

LIST OF FIGURES

Figure		Page
1	Viscous hydroplaning of smooth truck tire at 40 mph (64 km/h) in 0.08 in. (2 mm) of water (free rolling).	5
2	Sketch of bow wave in front of contact area of pneumatic tire at low vehicle speeds.	5
3	Bow wave--vehicle speed 20 mph (32 km/h).	6
4	Disappearance of bow wave-60 mph (97 km/h).	6
5	Estimated free rolling minimum full dynamic hydroplaning speed for passenger tires (conditions: relatively smooth surface, rounded footprint, and rated inflations and loads).	8
6	Forces in the footprint.	9
7	Bias tire at 100 mph (161 km/h) in 0.080 in. (2 mm) of water (free rolling).	10
8	Experimental tire at 40 mph (64 km/h) in 0.080 in. (2 mm) of water (free rolling).	12
9	Influence of tread pattern on lift coefficient.	13
10	Influence of tread patterns and tire type on lift coefficients.	15
11(a)	Wheel freely rolling on dry surface.	19
11(b)	Wheel not turning, 100% spin down condition.	19
12	Skid number trailer - drag link force measurement system and interface force diagram.	21
13	Influence of texture depths and water depth on transition point.	25
14	Data scatter for hydroplaning equation. (Data points at 10, 30 and 60% spin down).	26
15	Choice of expression for A to be used in Equation 2.	27

LIST OF FIGURES (continued)

Figure		Page
16	Influence of water depth on hydroplaning speed.	29
17	Influence of tread depth on hydroplaning speed.	30
18	Influence of tire pressure on hydroplaning speed.	31
19	Influence of texture depth on hydroplaning speed.	32
20	Deterioration of available friction with speed.	34
21	Possible range of hydroplaning speeds.	35
22	Skid numbers from 138 vs. spin down from 147.	38
23	Variation of water depth during rainmaker SN tests.	39
24	60 mph skid numbers vs. texture depths.	41
25	Resistance due to the displacement of water as a function of velocity for different water depths.	42
26	Effect of texture direction on skid numbers at calculated values of spin down.	43
27	Diagrammatic representation of water film flow over a sloping road surface.	46
28	Cross section of pavement macro asperities.	47
29	Textural profiles of the surfaces.	51
30	Rainfall runoff hydrograph.	53
31	Plot of water depths vs. variables for combined surfaces (points calculated from Equation 18).	54
32	Plot of water depths vs. variables of selected individual surfaces (observed or average data points).	56

LIST OF FIGURES (continued)

Figure		Page
33	Skid number vs. accident rate.	64
34	Variation of skidding accidents with vehicle speed and pavement condition.	65
35	Variation of wet road accidents with measured road surface friction.	66
36	Correlation of BPN (water fluid) with SN_{10}.	67
37	Compairson of total and injury accidents and coefficient of friction at 50 mph.	68
38	Variation of wet skidding accidents in traffic and passing lanes with stopping distance skid number (PSDN).	69
39	Interrelation among variables influencing automobile skidding on wet pavements.	71
40	The influence of water depth on tire cornering wet traction coefficient for tires of the same tread pattern in two different groove depths.	72
41	Variation of British Pendulum Number (BPN) with microtexture size and shape added to 4 mm glass spheres.	75
42	The increase in wet friction and tyre tread compound abrasion with increase in aggregate microtexture.	76
43	Effects of pavement texture depth (TXD), tire tread depth (TD) and water depth (WD) on the skid number (SN) as speeds of 20 mph to 60 mph, ASTM-14" tire, 24 psi tire inflation pressure.	80
44	Effect of tire inflation pressure on skid numbers (SN) for the ASTM-14" tire at minimum water depth (WD), minimum and maximum tire tread depth (TD), on low texture depth pavement (TXD).	82

LIST OF FIGURES (continued)

Figure		Page
45	Effect of tire inflation pressure on skid numbers (SN) for the ASTM-14" tire at maximum water depth (WD), minimum and maximum tire tread depth (TD) on low texture depth pavement (TXD), Texas A&M University Annex data.	83
46	Effect of tire inflation pressure on skid numbers (SN) for the ASTM-14" tire at minimum water depth (WD), minimum and maximum tire tread depth (TD) on high texture depth pavement (TXD), Texas A&M University Annex data.	84
47	Effect of tire inflation pressure on skid numbers (SN) for the ASTM-14" tire at maximum water depth (WD), minimum and maximum tire tread depth (TD) on high texture depth pavement (TXD), Texas A&M University Annex data.	85
48	Location of test sections, SH 6 in College Station, Texas.	91
49	Location of test sections, IH 10 near Van Horn, Texas.	92
50	Predicted vs. observed skid number for transverse textures using model No. 44.	100
51	Predicted vs. observed skid number for longitudinal textures using model No. 46.	101
52	Effect of vehicle speed on skid number for tread depth of 11/32 in. and texture depth of 0.030 in.	102
53	Effect of vehicle speed on skid number for tread depth of 2/32 in. and water depth of 0.00 in.	103
54	Maneuvers and cross-slope geometry.	107
55	Run No. 1 aligning torque.	109
56	Tangent path results.	113

Figure		Page
57	Run No. 9 results.	114
58	Aligning torque and required coefficient of friction vs. cross slope for lane change.	116
59	Approximate distributions of tire tread, tire pressure and surface texture for one state.	144
60	Probability of rainfall in Central Texas.	146
61	Curves of zero water depth for an intensity of 0.25 in./hr.	149
62	Curves of zero water depth for an intensity of 0.5 in./hr.	149
63	Curves of zero water depth for intensities of 0.75 and 1.0 in./hr.	150
64	Curves of zero water depth for an intensity of 1.5 in./hr.	151
65	Curves of zero water depth for an intensity of 2 in./hr.	151
66	Curves of zero water depth for an intensity of 4.0 in./hr.	152
67	Estimated acceleration boundaries as a function of SN_{40}.	155
68	Construction of available friction envelope.	156

LIST OF TABLES

Table		Page
1	Studies related to vehicle control forces available for comprehensive analysis.	17
2	Computed and observed hydroplaning speeds.	33
3	Descriptions of the surfaces studied for rainfall runoff.	50
4	Tabular representation of the relative defects of cross slope, texture, drainage path length and rainfall intensity on water depth.	58
5	Effect of cross slope and longitudinal gradient on drainage length.	60
6	Recommended AASHO guidelines for normal rural highway pavement cross slopes.	62
7	Pavement textural characteristics required for optimum tyre performance.	74
8	Surface finishes.	90
9	Skid measurements under simulated rainfall on SH 6 in College Station, Texas, October 1973. (ASTM E274 with 14 in. tire).	93
10	Skid measurements under simulated rainfall on IH 10 near Van Horn, Texas, October 1973. (ASTM E274 with 14 in. tire).	95
11	Regression models for skid numbers.	98
12	Cross slope geometry.	108
13	Summary of Tangent Path results.	111
14	Summary of Lane Change maneuvers.	115
15	Tabular summary of responses to the sag vertical curve drainage design questionnaire.	125
16	Tentative sag vertical curve drainage design criteria based on a survey of existing practice.	137

LIST OF TABLES (continued)

Table		Page
17	Probability of events and percentile rainfall.	148
18	Minimum texture and cross slope recommendations.	160
19	Tentative recommended polished stone values for various traffic volumes.	163

NOMENCLATURE

Symbol	Definition or Description	Dimension
A	- area of flow per unit width of pavement	ft.^{2} (m^2)
A	- (Chap. 2) the maximum of $(\frac{10.409}{WD^{0.06}} + 3.507)$ and $(\frac{28.952}{WD^{0.06}} - 7.817)TXD^{0.14}$	---
A	- (Chap. 6) the algebraic difference between two grades	---
A_G	- tire contact area	ft.^{2} (m^2)
A_1 - A_5	- constants used in Equation 37	---
B	- maximum width of contact patch	in. (mm)
b	- horizontal distance from center of rotation to the line of action of the vertical component of force	in. (mm)
BPN	- British Pendulum Number	---
BPN_ω	- relationship between SN_{10} and BPN ($BPN_\omega = 17.5 + 0.87\ SN_{10}$)	---
C_H	- lift coefficient	---
$C_{L,S}$	- hydroplaning lift coefficient	---
C_1 - C_6	- constants developed from Equation 37	---
D	- hydrodynamic drag force	lb. (kg)
DL	- drag link	---
d_i	- cross section depth of each groove as obtained dynamically on a wet glass plate	in. (mm)
e	- base of the natural logarithm	---
F,F'	- horizontal force (tire-surface friction)	lb. (kg)
F_v	- vertical ground force	lb. (kg)
F_{SW}	- total tangential force	lb. (N)
F_y	- lateral force	---
g	- gravitational acceleration	ft./sec.^{2} (m/s^2)

Symbol	Definition or Description	Dimension
HD	- hydrodynamic drag	lb. (kg)
I	- rainfall intensity	in./hr. (mm/h)
I	- (Chap. 2) polar moment of inertia of the wheel about its axis of rotation	---
K	- constant	
K	- (Chap. 3) used for a velocity relationship	
K	- (Chap. 6) design variable equal to the length of the curve (L) divided by the algebraic difference between the two grades (A)	---
k	- hydraulic conductivity	ft./sec.(m/s)
K_1	- weighting factor to consider lateral grooving and blading	---
L	- drainage length	ft. (m)
L	- (Chap. 1) length of the tire footprint region	ft. (m)
L	- (Chap. 3) pavement width when used in equation $L_f = L \frac{S_3}{S_1}$	ft. (m)
L	- (Chap. 6) length of the curve	---
L_f	- length of flow path ($L_f = L \frac{S_3}{S_1}$)	ft. (m)
m	- exponent equal to 1/3 for laminar flow	---
N_F	- Froude number	---
N_R	- Reynold's number	---
n	- rib number	---
n	- (Chap. 3) exponent equal to 1/3 for laminar flow	---
O,o	- moment center	---
P,p	- tire pressure	psi (kPa)
P_{IN}	- tire pressure in kPa	kPa
Q	- wheel load	lb. (kg)
q	- discharge per unit width of pavement	$ft.^3/sec.$ (m^3/s)
R,R'	- resultant force	lb. (kg)

NOMENCLATURE (continued)

Symbol	Definition or Description	Dimension
R^2	- coefficient of determination (correlation coefficient squared)	---
r	- correlation coefficient (Chaps. 2 & 4)	---
r	- tire radius	in. (mm)
S	- cross (transverse) slope	in./ft. (%)
S	- cross (transverse) slope (chaps. 3 & 7)	ft./ft. (%)
S_1	- cross (transverse) slope	ft./ft. (%)
S_2	- longitudinal gradient	ft./ft. (%)
S_3	- slope of resultant flow path	ft./ft. (%)
SD	- spin down	%
SDT	- spin down trailer	---
SE	- standard error	---
SN_x	- skid number where x is in mph	---
SNT_{dl}	- skid number trailer - drag link	---
SNT_t	- skid number trailer - torque	---
T	- resultant torque	in.-lb. (m-kg)
t	- water film thickness	in. (mm)
T_a	- air drag torque	in.-lb. (m-kg)
T_b	- bearing drag torque	in.-lb. (m-kg)
TD	- tread depth	32nd of in. (mm)
TX	- maximum texture depth	in. (mm)
TXD	- texture depth	in. (mm)
ΔT_{SF}	- amount of time necessary for the water film thickness to be sufficiently reduced so that contact is made between the tread rubber and the pavement microasperities	sec.
T_{SW}	- steering wheel torque	in.-lb. (Nm)
UGC	- unit tire groove capacity	
	$\left(UGC = \frac{K_1}{W_t} \sum_{i=1}^{n} w_i d_i \right)$	in. (mm)

xvi

NOMENCLATURE (continued)

Symbol	Definition or Description	Dimension
V, V_p	- hydroplaning speed	mph (km/h)
V	- (Chap. 3) mean velocity of flow	mph (m/s)
V_H	- limiting hydroplaning speed	m/s
W, W'	- vertical component of the resultant force	lb. (kg)
WD	- water depth	in. (mm)
WD_u	- water depth above the bottom of the texture	in. (mm)
W_T	- effective width of tread	in. (mm)
w_i	- cross section width of each groove as obtained dynamically on a wet glass plate	in. (mm)
α	- angular deceleration of the wheel	ft./sec.^{2} (m/s^2)
α	- (Chap. 3) constant used for expressing a relationship of depth	---
β	- constant used for expressing a relationship of slope	---
θ	- bow wave angle	degree
μ	- coefficient of friction	---
ν	- kinematic viscosity	ft.^{2}/sec.^{2} (cm^2/s^2)
ρ	- fluid density	slug/ft.^{3} (kg/m^3)
ω_d	- rotational velocity of the rolling wheel when on a dry surface	mph (km/h)
ω_w	- rotational velocity of the wheel after spinning down due to contact with a flooded pavement	mph (km/h)

GENERAL INTRODUCTION

What is this phenomenon of hydroplaning? Is it some calamitous event that suddenly takes hold of an unwary traveler destroying his capability to control the vehicle; or is it a gradual, smoothly progressing deterioration of available friction progressing to a level where the vehicle becomes unsteerable; or is it all of these things? How can an engineer know if a roadway will allow hydroplaning to occur; and what can be done to overcome the condition? Answers to these and other related questions were sought in the literature in the early phase of this study, a literature review that resulted in answers to many of the important questions.

Some of the earliest investigations and technical reports on hydroplaning came from NACA and its successor NASA and were primarily concerned with hydroplaning of aircraft during landings. In this connection, the U. S. Army Air Corps and its successor U. S. Air Force also did valuable work. Later the Road Research Laboratory in Great Britain began investigations related to automobiles. Concurrent with this research, the Americans and Germans studied tires and road surfaces to seek their own answers. Most recently, the Highway Research Board, now the Transportation Research Board, the National Cooperative Highway Research Program and the FHWA have encouraged and are financing studies related to tire-pavement interaction and hydroplaning, studies that should bring the "state of the art" to a most respectable level within the next few years.

Although the extensive bibliography developed as a part of this study has been surveyed for background material useful in the accomplishment of the different tasks relating to the subject of hydroplaning, it is likely that further work will reveal other material now held as privileged information. Particular reference is made to various rubber companies now engaged in optimizing rubber compounds and tread designs used on radial tires. Several now claim a tread that removes water for better traction. Some significant advances in this area may soon be revealed. However, there is no need to wait for these breakthroughs before progress can be made in lowering the probability of hydroplaning. The information now available, although not complete, is certainly advanced enough to justify some significant implementation activities, as will be shown in detail in the main body of this report.

The following subjects summarize briefly the main areas where good information has been found or developed:

1. The definition of full dynamic hydroplaning conditions.
2. The definition of changes in available friction as full hydroplaning is approached as a function of a variety of textural conditions, both random and patterned.
3. The definition of road surface geometry and how it affects the accumulation of water under various environmental conditions.

4. The probability of different conditions relating to hydroplaning such as tire condition and rainfall intensity.
5. The influence of various pavement cross slopes on driver work load and performance, as indicated by vehicle accelerations and friction demand levels.
6. The treatments given sag vertical curves by state highway and transportation agencies.

In Chapter VII preliminary indications of the range and frequency of important factors such as tread depth, tire pressure, surface texture, cross slope and rainfall intensity are given and used to develop tentative design criteria. Curves are developed,(Figures 61 through 66) based on a capability to predict water depth from given geometric and environmental conditions which can be used to determine appropriate combinations of pavement cross slope and texture depth during periods of specified rainfall intensity.

The implementation of interim criteria and recommendations for surface texture and pavement cross slope to minimize hydroplaning has the potential for a tremendous impact on the unacceptably high wet weather accident toll. The information contained in this report justifies this action. Further, it is hoped that the means used to accomplishd this reduction may be extended to other nations that are now experiencing similar problems.

CHAPTER I

THE PHENOMENON OF HYDROPLANING

There have been almost as many definitions of hydroplaning as there have been researchers on the subject; but when all the linguistic frills are cut away, what remains is the separation of the tire from the road surface by a layer of water. On a microscopic scale all operational conditions may involve some degree of partial hydroplaning as long as there is any significant amount of water present. On a macroscopic scale, however, this zone can be defined as occurring during those operational conditions when there is some significiant degree of penetration of a water wedge between the tire and pavement contact area.

Hydroplaning of pneumatic tired vehicles has been divided into three categories by A. L. Browne (1). These categories are:

1. viscous hydroplaning,
2. dynamic hydroplaning and
3. tire tread rubber reversion hydroplaning.

Viscous and dynamic hydroplaning are the important types of hydroplaning insofar as passenger vehicles are concerned since tire tread reversion hydroplaning occurs only when heavy vehicles such as trucks or aircraft lock their wheels when moving at high speeds on wet pavements with macro- but little microtexture.

Viscous hydroplaning may occur at any speed and with extremely thin films of water. Browne (1) states that wet friction hydroplaning occurs only on surfaces where there is little microtexture. A thin film of water remains between the tire and pavement since there is insufficient pavement microtexture to promote the breakdown of the water film. For pavements exhibiting only a small amount of microtexture, Browne (1) gives the following expression for limiting viscous hydroplaning speed:

$$V_H \geq L/\Delta T_{SF}$$

Where

V_H = minimum speed necessary for viscous hydroplaning
L = the length of the tire footprint region
ΔT_{SF} = the amount of time necessary for the water film thickness to be sufficiently reduced so that contact is made between the tread rubber and the pavement microasperities.

Browne also states that tire inflation pressure determines whether the viscous form of hydroplaning will persist at moderate to high speeds. The greater the tire inflation pressure, the greater will be the probability of high speed viscous rather than dynamic hydroplaning.

R. W. Yeager (2) states that when removal of the water film from beneath the tire footprint is resisted by internal friction within the tire fluid layer, the resulting action is referred to as viscous tire hydroplaning. He further states that the formation of the fluid layer can occur on smooth or macrotextured surfaces at any speed above 20 mph (32 km/h). This type of hydroplaning is shown in Figure 1, where a thin film of water has penetrated the complete tire footprint area of a smooth truck tire at 40 mph (64 km/h) and has separated the tire from the surface. This photograph is made (either by still or movie camera) from beneath a glass plate as the tire is moved over the glass plate. A fluorescein dye solution is added to the water to indicate flow and define tread to surface contact with the glass plate. A question of terminology may warrant discussion at this point. Browne and Yeager consider a rather pure form of viscous hydroplaning where a thin film of water completely separates the tire from the very smooth surface. However, a much more common occurrence is when the tire is only partially separated from the surface by thin films of water resulting in a significant decrease in friction. Thus partial viscous hydroplaning may be the predominate factor causing low values of wet friction. The term viscous hydroplaning might then be considered the cause while reduced wet friction is considered the result.

Horne and Dreher (3) state that a bow wave forms in front of the tire for all speeds less than the hydroplaning speed. As the vehicle speed (v) increases, the bow wave angle (Θ) decreases (see Figure 2) until at some speed approximating the hydroplaning speed the bow wave completely disappears. The existence of a bow wave is shown in Figure 3 where the test vehicle speed is 20 mph (32 km/h). On Figure 4 where the test vehicle speed is 60 mph (97 km/h), the bow wave has disappeared. Browne (1) states that dynamic hydroplaning occurs when the amount of water encountered by the tire exceeds the combined drainage capacity of the tread pattern and the pavement macrotexture. It occurs in deep fluid layers where fluid initial effects are dominant. The pressures built up on the tire surfaces due to its collision with the stationary fluid are sufficient to buckle the tire tread surface inward and upward from the pavement causing a progressive penetration (with increasing speed) of the fluid film from front to rear of the footprint region.

Horne and Joyner (4) give the following formula for the approximate speed at which a tire will dynamically hydroplane as:

$$V_H = 1.8 \sqrt{P_{IN}}$$

where

V_H = the limiting hydroplaning speed in m/s
P_{IN} = the tire inflation pressure in kilo Pascals (kPa)

Horne and Dreher (3) give an equation for tire hydroplaning speed as:

(after R. W. Yeager (2))

Figure 1. Viscous hydroplaning of smooth truck tire at 40 mph (64 km/h) in 0.080 in. (2 mm) of water (free rolling).

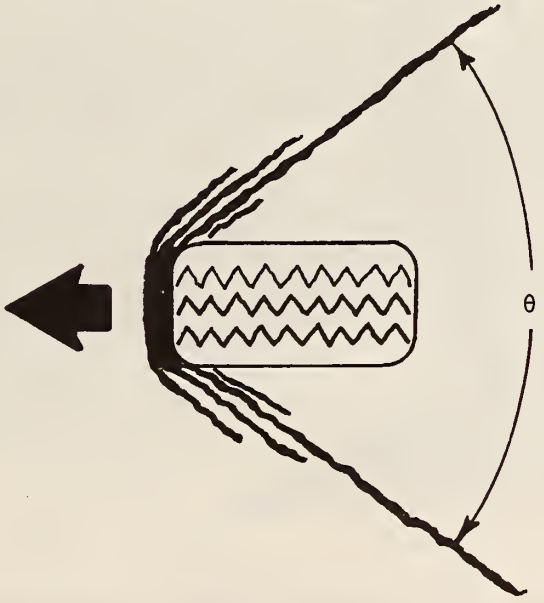


Figure 2. Sketch of bow wave in front of contact area of pneumatic tire at low vehicle speeds.

Figure 3. Bow wave--vehicle speed 20 mph (32 km/h).

Figure 4. Disappearance of bow wave--60 mph (97 km/h).

$$V_p = 0.592 \left[\frac{F_v}{A_G} \frac{288}{C_{L,S}\ \rho}\right]^{1/2}$$

where

V_p = hydroplaning speed
A_G = tire contact area
ρ = fluid density
F_v = vertical ground force
$C_{L,S}$ = hydroplaning lift coefficient

The following assumptions are then made:

1. F_v/A_G may be approximated by the tire pressure:
2. the fluid density is that of water; and
3. $C_{L,S} \simeq 0.7$.

The equation then reduces to:

$$V_p = 10.35 \sqrt{p}$$

where

V_p = hydroplaning speed in mph ((km/h)/1.609)
p = tire inflation pressure in psi (kPa/6.895)

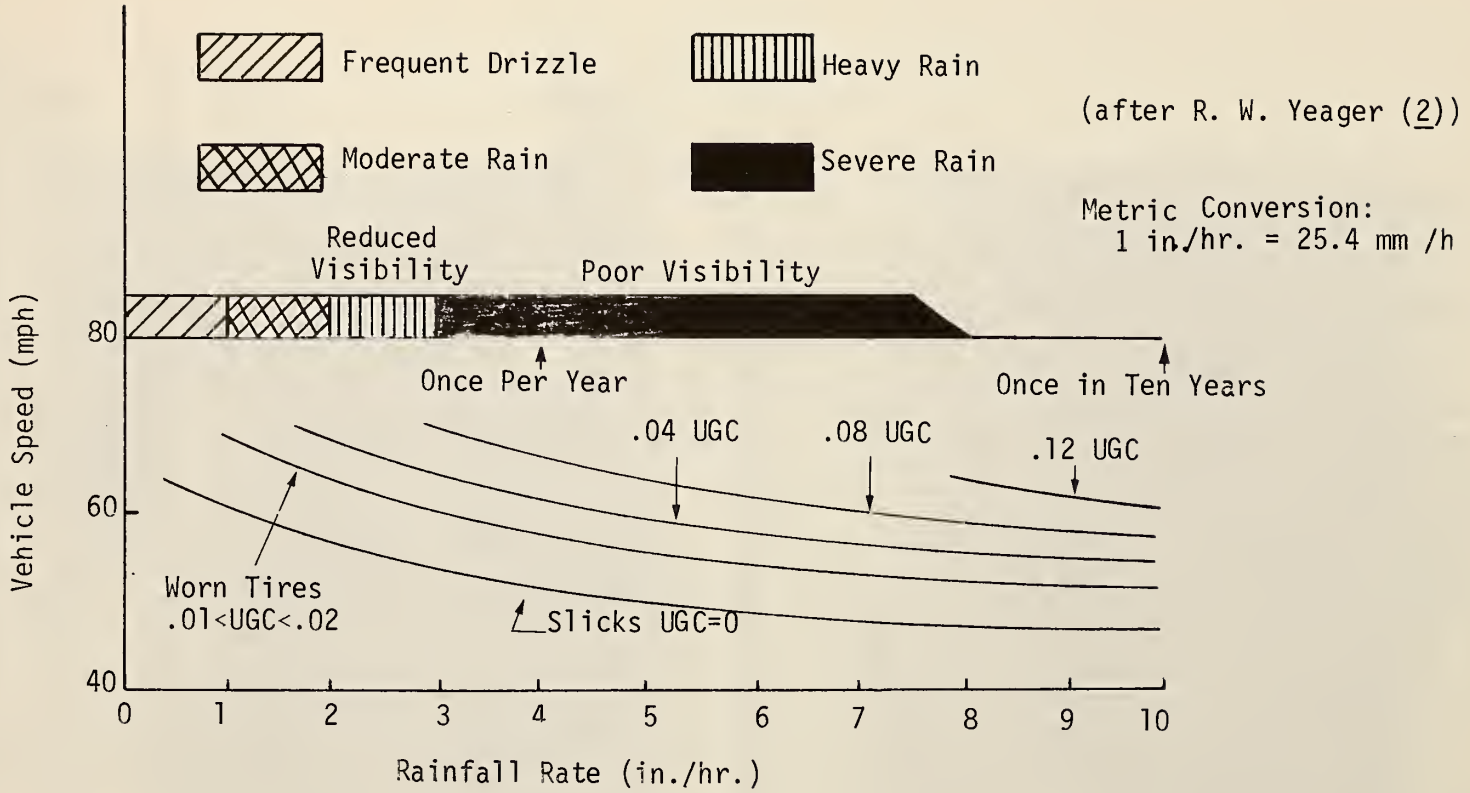
R. W. Yeager (2) has developed a figure for estimating full dynamic hydroplaning speed for passenger tires travelling on relatively smooth surfaces based on the UGC. See Figure 5. The UGC is the unit tire groove capacity and is defined by:

$$UGC = \frac{K_1}{W_T} \sum_{i=1}^{n} w_i\ d_i$$

where

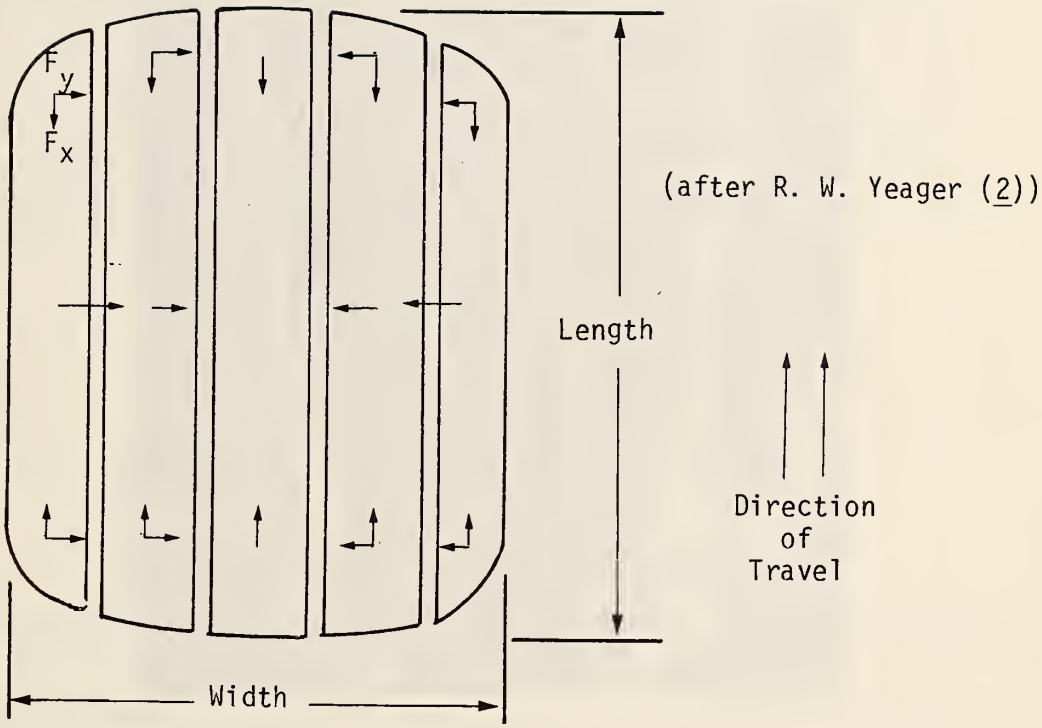
W_T = effective width of tread
w_i and d_i = individual cross section widths and depths of each groove as obtained dynamically on a wet glass plate
n = rib number
K_1 = weighting factor to consider lateral grooving and blading

Figure 6 shows the forces developed in the tire footprint area as the tire rotates. The lateral force, F_y, is greatest for bias tires and decreases rapidly when a belt is applied such as with a radial tire. On a dry surface the tire friction is high enough to resist closure. On wet surfaces, however, the low friction coefficient allow groove closure to occur which greatly reduces the UGC of the tire. Figure 7 shows groove closure for a bias tire. This closure tendency is not pronounced in radial tires.



Water Depth (in.)											
	.030	.045	.057	.067	.075	.082	.088	.093	.097	.101	2.1% Slope, 11 Ft. Drainage Length
	.045	.060	.072	.082	.092	.102	.110	.117	.124	.130	1.56% Slope, 22 Ft. Drainage Length

Figure 5. Estimated free rolling minimum full dynamic hydroplaning speed for passenger tires (conditions: relatively smooth surface, rounded footprint, and rated inflations and loads).



(after R. W. Yeager (2))

Figure 6. Forces in the footprint.

(after R. W. Yeager (2))

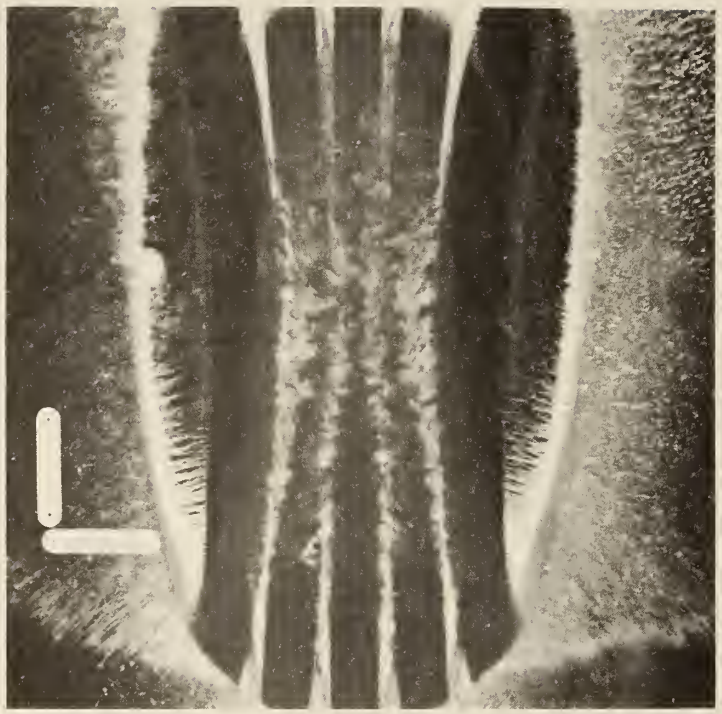


Figure 7. Bias tire at 100 mph (161 km/h) in 0.080 in. (2 mm) of water (free rolling).

Figure 8 shows fluid flow in longitudinal grooves in a tire footprint region. Fluid flow is confined to the forward portion of the tire footprint and to the grooves. Twin vortices are formed at the forward edge of the footprint.

Browne (1) has presented a mathematical model for hydroplaning that allows for the presence of tire surface deformation. This model indicates that tire deformation is an essential element for dynamic hydroplaning of pneumatic tires.

One of the most significant studies of the influence of tread design on hydroplaning speed was conducted by Gengenbach (5). In this study he compared seven conventional bias ply tires with a variety of tread patterns designed and cut into the tire so that the total volume of grooves was constant (groove area ratio was 20%). These patterns are shown in Figure 9 (patterns A through F). For comparison purposes an ungrooved tire, G, is included in the figure. The lift coefficient, C_H, is given by the bar graph. This lift coefficient was derived by tests and was used in the equation,

$$V = 508 \sqrt{\frac{Q}{B\ t\ C_H}} \qquad (1)$$

where

V = hydroplaning speed in km/h
Q = wheel load in kg
B = maximum width of contact patch in mm
t = thickness of water film in mm

which gave good agreement with test data for water depths from 0.3 to 4 mm (0.01 to 0.16 in.).

One conclusion not drawn by Gengenbach seems appropriate here. While for bald tires hydroplaning was observed at water depths as low as 0.3 mm (.01 in.), for treaded tires no hydroplaning data have been found in the report for water depths less than 2 mm (.08 in.). Evidently, for the low values of texture used in these tests (80 grit surface probably corresponds to a texture between .005 and .008 in. (.13 and .20 mm) depending on imbedment of the grains in the retaining cement), treaded tires did not hydroplane at water depths less than .08 in. (2 mm). Since the nominal tread depth of the tires was 0.28 in. (7 mm), it appears that there was space for almost all the water in the contact patch within the grooves. Considering the 20% groove area ratio and assuming 80% of the water needs to be accommodated by grooves (20% escapes around the circumference), the relative volumes can be checked to see if this could be true.

(after R. W. Yeager (2))

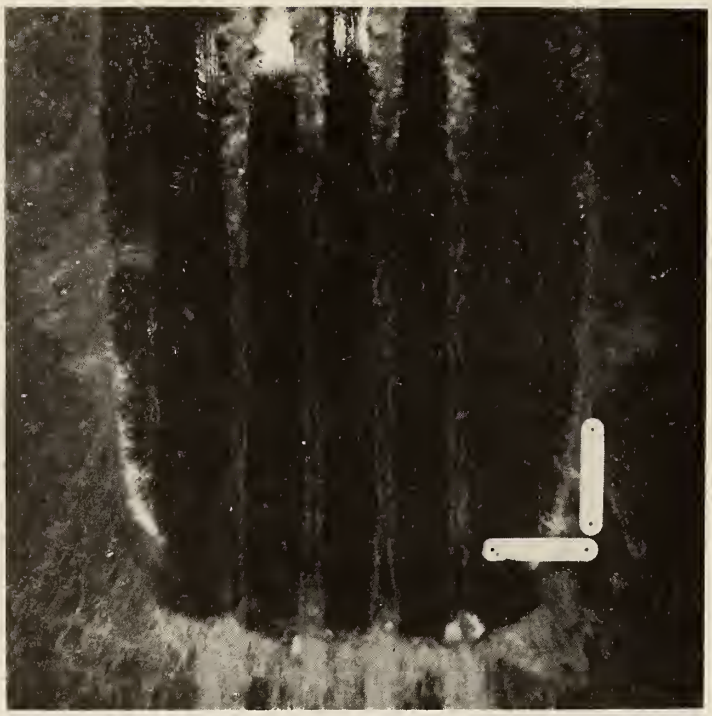


Figure 8. Experimental tire at 40 mph (64 km/h) in 0.080 in. (2 mm) of water (free rolling).

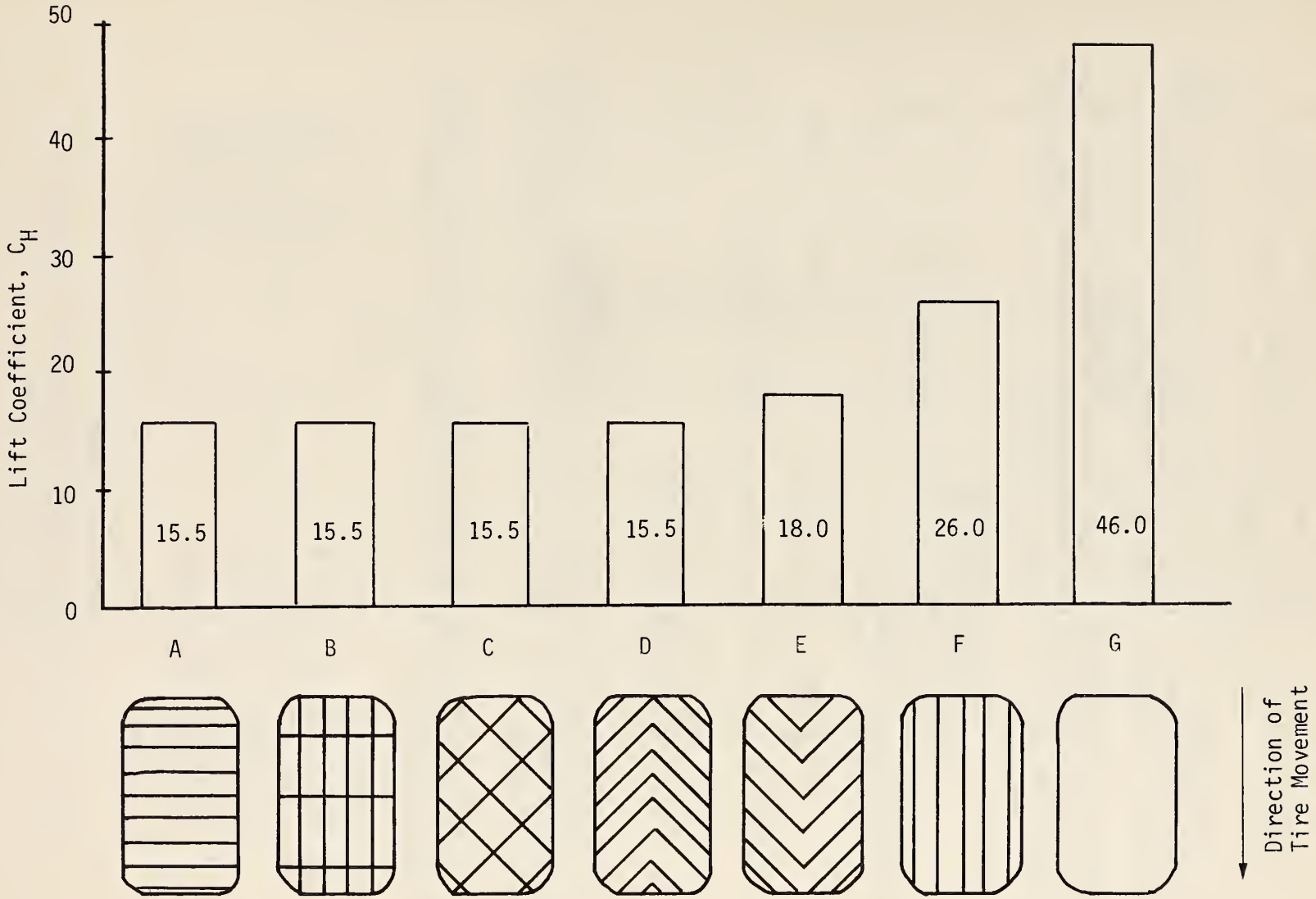


Figure 9. Influence of tread pattern on lift coefficient.

Volume to be accommodated = (.08 in.) (contact patch area) (0.8)
= .064 (contact patch area)
Volume available = (0.28 in.) (contact patch area) (0.2)
= .056 (contact patch area)

The rough correspondence of .056 and .064 indicates the likelihood that this was the case. That is, there is room enough in the grooves to accommodate most of the water under the contact patch.

Returning to the subject of tread pattern influence, Equation 1 can be applied to determine corresponding values of hydroplaning speed. For these purposes the following values are chosen:

$Q = 300$ kg (660 lbs.)
$B = 100$ mm (3.94 in.)
$t = 2.54$ mm (0.1 in.)

For tread patterns A through D,

$$V = 508 \sqrt{\frac{300}{(100)\ (2.54)\ (15.5)}} = 140 \text{ km/h (87 mph)}$$

Use of C_H values of 18, 26 and 46 gives the following speeds:

$V_E = 81$ mph (130 km/h)
$V_F = 68$ mph (109 km/h)
$V_G = 52$ mph (84 km/h)

Thus it appears that a 20 mph range of hydroplaning speed from 68 to 88 mph (32 km/h range from 109 to 142 km/h) can result from tread pattern, and a 36 mph range from 52 to 88 mph (58 km/h range from 84 to 142 km/h) can result from tread wear.

Further comparison of the lift coefficients for commercial tires is given by Figure 10, also emphasizing the influence of tread pattern and wear on hydroplaning speeds.

Figure 10. Influence of tread patterns and tire type on lift coefficients.

CHAPTER II

EMPIRICAL INDICATIONS OF HYDROPLANING

In addition to the findings of research efforts already cited, the authors are fortunate in that several research projects relating either directly or indirectly to hydroplaning have just been completed. These projects are proving to be highly useful in the development of a quantitative definition of partial hydroplaning conditions. They are summrized in Table 1 and include locked wheel friction measurements and vehicle handling tests, each under a number of pavement tire and water conditions.

The study directly related to hydroplaning (147) is an extensive empirical study of tires, pavements and water depths which compare closely to real highway conditions. The water depths are all over 0.1 in. (2.5 mm) and therefore extend the information available to the larger water depth conditions. These tests show a range of full dynamic hydroplaning speeds from 45 to 77 mph (72 to 124 km/h).

Interpretation of Data from Different Test Mechanisms

In any effort devoted to interpretation of test data it is essential to determine what, exactly, the test mechanism is measuring. The problem is compounded when several different test mechanisms are used to obtain data which should be interrelated. In the case of data developed by TTI as an indication of tire-pavement wet friction and hydroplaning, there are three mechanisms involved. Each is a towed trailer, but two give a measure of the shearing force between the tire and the wet or flooded pavement while one measures the spin down of the test wheel. The latter is an indication of changes in the magnitude and orientation of the total force acting on the tire-wet pavement interface. In order to interpret data from each of these mechanisms the following discussion will show what each is measuring. The test mechanisms, designated SDT (Spin Down Trailer), SNT_{dl} (Skid Number Trailer - drag link) and SNT_t (Skid Number Trailer - torque), will be discussed in the order shown.

Spin Down Trailer

In developing the body of data on HPR-147 the indication of hydroplaning which was used was the spin down of a test wheel when moved in a freely rolling condition across a section of pavement covered by a significant layer of water. Spin down is usually given in percent as dictated by the following equation:

$$SD = \frac{\omega_d - \omega_w}{\omega_d} (100)$$

Table 1. Studies related to vehicle control forces available for comprehensive analysis.

HPR STUDY NUMBER	TEST MECHANISM	PAVEMENT TYPE	WATER APPLICATION	DEPENDENT VARIABLE
138	ASTM Skid Trailer	Asphalt & PCC Concrete	Rainmaker	SN Drag Link
141	ASTM Skid Trailer	PCC Concrete	Rainmaker	SN Drag Link & Torque
147	Spin Down Trailer	Asphalt & PCC Concrete	Trough	Spin Down, %
163	Automobiles	Asphalt & PCC Concrete	Sprinklers & Flow Pipes	Long. & Trans. Accelerations

where

ω_d = rotational velocity of the rolling wheel when on a dry surface in mph ((km/h)/1.609)

ω_w = rotational velocity of the wheel after spinning down due to contact with a flooded pavement in mph ((km/h)/1.609)

Figure 11 will be used to illustrate the forces acting on the tire in two extreme conditions: (1) freely rolling on a dry surface and (2) the condition of 100% spin down when the tire is sliding across a water surface with little, if any, direct contact between tire and pavement surface. At 100% spin down, the rotational velocity of the tire is equal to zero.

In the freely rolling condition with the rotational velocity a constant, K, a summation of moments about the center of the wheel is given by

$$T_a + T_b = Fr$$

when the vertical component, W, of the resultant force, R, is assumed to pass through the moment center, O, or to pass so close to it that the torque induced by W is negligible. Thus, in order for the tire to rotate at the constant value of ω, without significant contact zone slip, only enough friction needs to be developed to overcome the retarding effects of rolling resistance, which includes bearing friction and air drag. Thus, the value of F is small with respect to W. Up to 60 mph (97 km/h) the rolling resistance is generally less than 2% of W (6).

In the 100% spin down condition, a condition considered to be indicative of full dynamic hydroplaning, the resultant force, R, changes to some degree in magnitude, but the major change is in its line of action. These changes are caused by (1) the destruction of friction between tire and pavement surface, (2) the development of pressure on the tire in the region of contact with the water wedge and (3) the development of a hydrodynamic drag force on the tire-water interface.

The resultant force, R', can still be resolved into two components, the vertical, W', equal to the weight supported by the wheel and the horizontal force, D + F'. Hydrodynamic drag, D, is due to the inertia of the water over and through which the tire is sliding. F' is some small remaining value of tire-pavement friction. Now the moment summation gives

$$W'b = (D + F')r.$$

Thus the change in line of action of R' produces a counterclockwise torque due to W which counteracts the torque due to D. Stated another way the line of action of R' moves to a position which includes the moment center, O, resulting in negligible torque on the wheel.

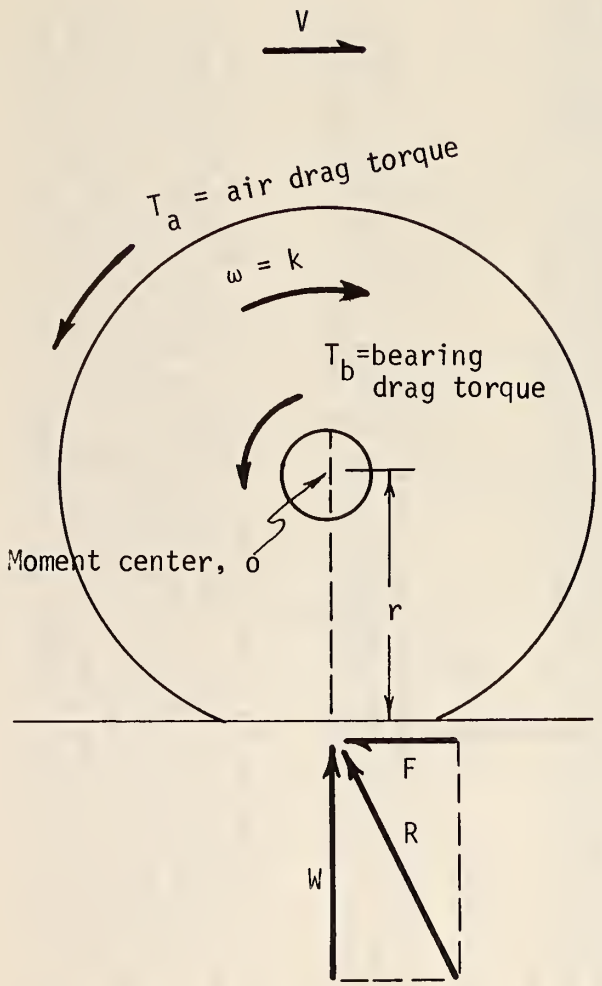


Figure 11(a). Wheel freely rolling on dry surface.

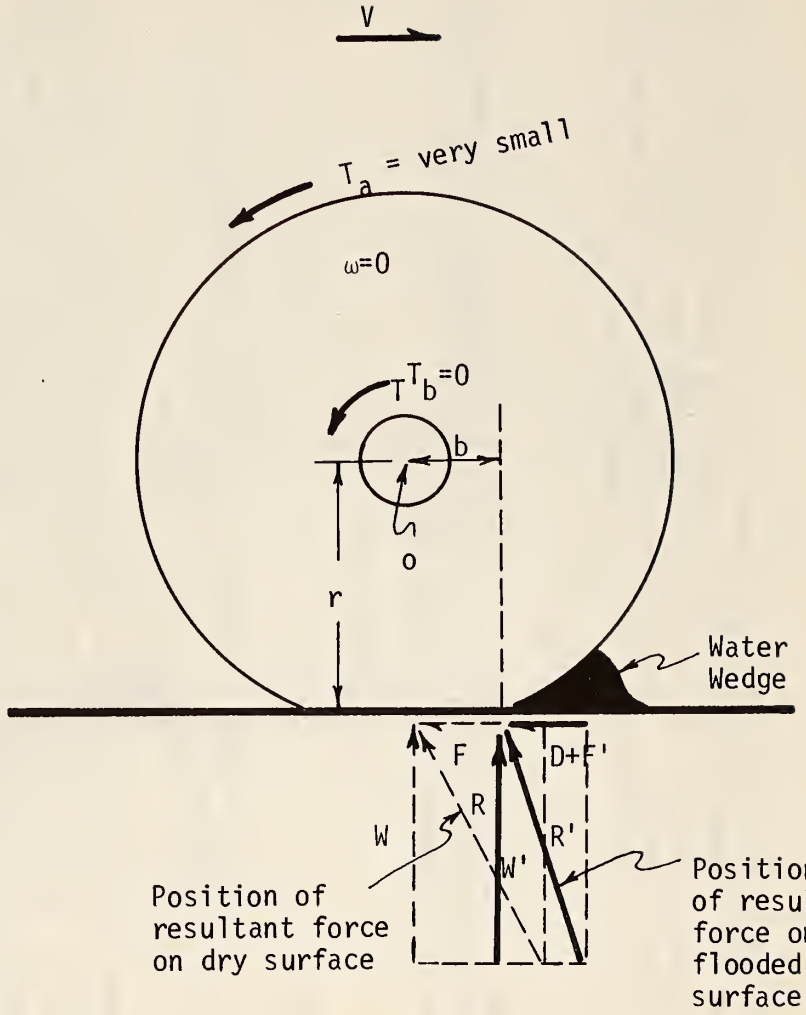


Figure 11(b). Wheel not turning, 100% spin down condition.

Now it is seen that 100% spin down is not necessarily a magic indicator of full dynamic hydroplaning, a condition which may occur at spin down values of 10%, 50% or even 101% (tires have been observed to reverse direction of rotation). The value of spin down is only indicative of a fine balance of different forces and elements of drag within the equation

$$T_a + T_b + Wb = Dr + F'r$$

which may or may not include the F' term, F' being some small remaining value of tire-surface friction. If the wheel is in the process of spinning down, the appropriate equation would be

$$T_a + T_b + Wb = Dr + F'r + \propto I$$

where

I = polar moment of inertia of the wheel about its axis of rotation
$\propto$ = angular deceleration of the wheel

Summarizing, test mechanism SDT measures wheel rotation velocity as it is drawn over a flooded pavement in a freely rolling condition. Wheel spin down is caused by changes in the magnitude and position of the line of action of the resultant force, R. These changes are caused by the development of a water wedge producing forces on the lower front of the tire and the destruction of significant values of F, the force due to available tire-surface friction. It is the destruction of F that is the most important result of hydroplaning.

Further, it has not been determined that F must be fully destroyed in order for spin down to occur. However it does seem apparent that F' must be very small in order for spin down to occur and also that larger values of spin down should indicate progressively smaller, probably insignificant values of F'.

Skid Number Trailer - drag link

All of the data developed under HPR-138 and a portion of the data developed under HPR-141 were obtained using a skid trailer with a "drag link" type force measurement transducer. This mechanism is diagrammed in Figure 12.

A test is conducted by drawing the trailer over a wet or flooded surface and locking the brake of the test wheel so that the wheel ceases rotation. The wheel is freely rotating prior to brake lock. Once the lockup is achieved the resultant horizontal force acting on or very close to the tire-surface interface is transmitted through the linkage from the spring to linkage A to the drag link, DL, to the cantilever force transducer. Note that the force transmitted through the DL to the cantilever

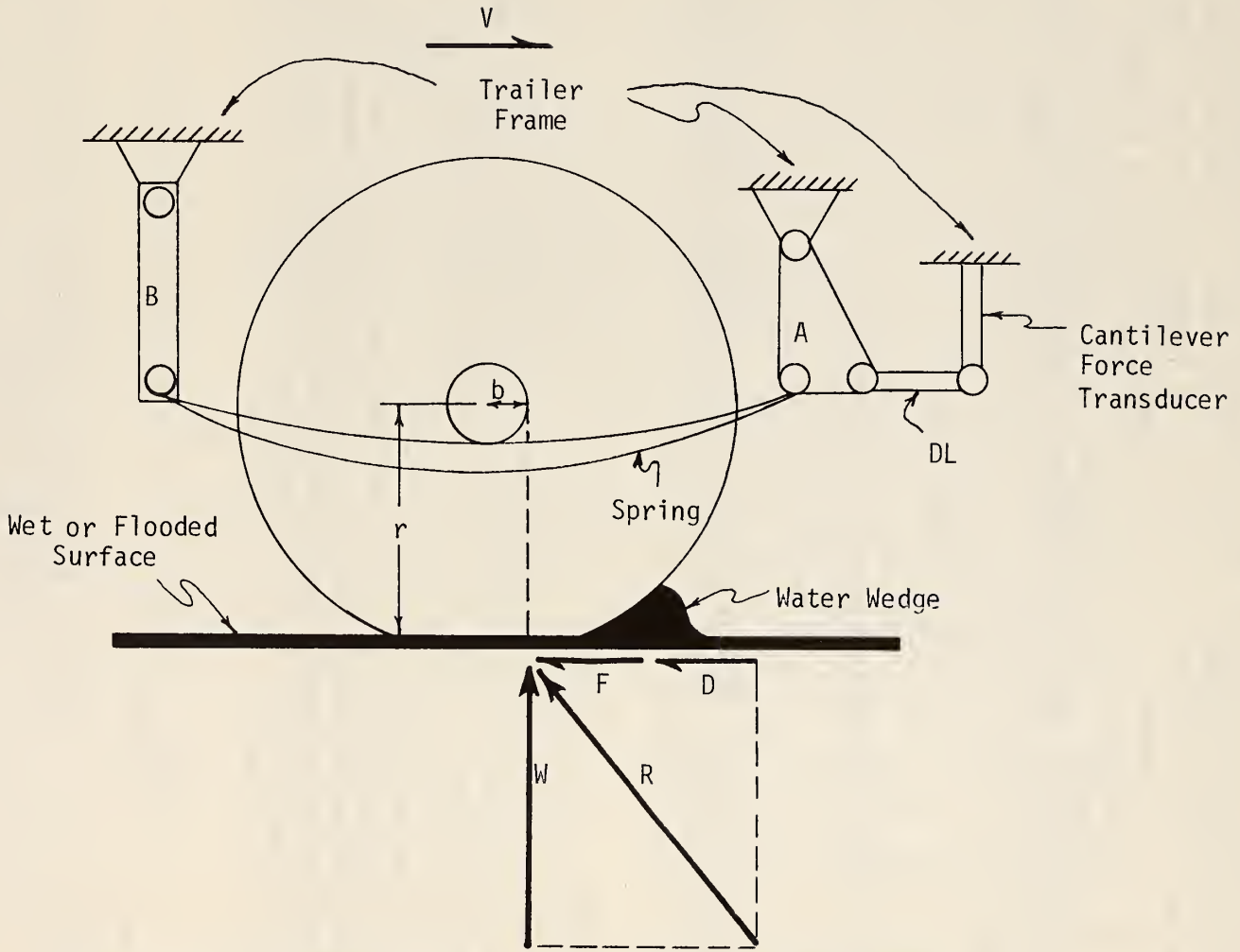


Figure 12. Skid number trailer - drag link force measurement system and interface force diagram.

is not sensitive to either the magnitude or location of the vertical component of the resultant force, R. Therefore the mechanism would still read the horizontal component of the resultant even if R passed through the center of the wheel causing no resultant torque. If the wheel lost contact with the pavement, as in full dynamic hydroplaning, there would still be a hydrodynamic drag force, D, which would indicate a certain value of skid number. For any condition over a surface and through a significant layer of water the component D, hydrodynamic drag, is contributing to the indication of SN. If the terminology of the diagram shown is used

$$SN = \frac{D + F}{W} (100).$$

For full dynamic hydroplaning, where F becomes zero,

$$SN = \frac{D}{W} (100) .$$

Summarizing, the SN values developed by mechanism SNT_{d1} include the effect of hydrodynamic drag, and therefore overestimate the force developed by tire-surface friction. A significant SN would correspond to 100% SD.

Skid Number Trailer - torque

The later part (chronologically) of the data developed under HPR-141 was obtained using a skid trailer with a torque transducer located at the center of the wheel. Tests are run in the same way as explained under SNT_{d1} but in this case there is a definite sensitivity to changes in the magnitude and position of the line of action of the vertical component of force W (see Figure 12). The force transducer is sensitive to the resultant torque on the trailer test wheel and is located symmetrically with respect to the center of rotation of the test wheel. If the line of action of W does not coincide with the center of the wheel this resultant torque, T, is given by

$$T = (D + F)r - Wb$$

Skid number is then calculated as

$$SN = \frac{T/r}{W} (100)$$

$$= \frac{(D + F)r - Wb}{rW} (100)$$

The indicated value of SN is then increased by the influence of D, hydrodynamic drag, but decreased by the forward shifting of the line of action of W due to development of pressures on the tire from the water wedge. If a hydroplaning condition corresponding to 100% spin down existed (as observed by SDT), where the resultant force, R, passed through the center of the wheel, zero torque would be indicated resulting in a zero value for SN.

Summarizing, the SN values developed by mechanism SNT_t include the influence of two extraneous factors which act in opposition to each other. Therefore their effect is cancelled to some extent, resulting in a better estimate of the force developed by tire-surface friction. A zero SN would correspond to 100% SD, but not necessarily to zero friction.

Interpretation of Data from HPR Studies 147, 138 and 141

In interpreting the meaning of hydroplaning data developed under HPR-147, it was concluded that significant control forces between the tire and the pavement did not exist at high values of wheel spin down, i.e., over 50%. Further, the difference between 10% spin down and 50% spin down when expressed in mph (test vehicle speed) was so small that 10% spin down was chosen as the indicator of hydroplaning. This does not imply that full dynamic hydroplaning was present at 10% spin down because an exact way has not been achieved to interpret percent spin down for the amount of tire contact patch touching the pavement or for vehicle control forces. Before this interpretation could be attempted it was necessary to develop a general equation that would allow the vehicle speed to be predicted so that a specific value of SD would occur as a function of the variables denoting test conditions. These parameters were as follows:

1. V, vehicle speed in mph ((km/h)/1.609)
2. TD, tread depth in 32nds of an in. (mm/25.4)
3. TXD, texture depth in in. (mm/25.4), silicone putty method
4. WD, water depth in in. (mm/25.4)
5. P, tire pressure in psi (kPa/6.895)
6. SD, spin down %

Based on data from approximately 1,300 tests and the curves of SD vs. V which fit these data for specific test conditions, 1,038 points were selected at SD values of 10, 32 and 60%. When a correlation was developed using all these data it was found that the equation was not appropriately sensitive to texture depth. This equation was:

$$V = 5.95\ SD^{0.04}\ p^{0.28}\ (TD + 1)^{0.05}\ TXD^{0.01}\ \left[\frac{1.82}{WD^{0.01}} + 1\right]$$

The reason for this lack of sensitivity was found in the nonuniform distribution of data over the texture range from 0.001 in. to 0.150 in. (.025 to 3.81 mm). It was observed that 80% of the data was within a texture range of 0.001 to 0.033 (.025 to .838 mm); and that within this range there was indeed little sensitivity to texture. The preponderance of data within this range was forcing a conclusion of insensitivity on the range of textures above 0.033 inch (.84 mm). To overcome this problem, a "two-step constrained select regression" was developed which resulted in the following equation:

$$V = SD^{0.04}\ P^{0.3}\ (TD + 1)^{0.06}\ TXD^{a}\ \left[\frac{b}{WD^{0.06}} + c\right] \qquad (2)$$

for TXD $\leq$ 0.033 in. (.84 mm), use a = 0, b = 10.409 and c = 3.507

for TXD > 0.033 in. (.84 mm), use a = 0.14, b = 28.952 and c = -7.815

Note that for texture depths below 0.033 in. (.84 mm) the data do not show any sensitivity (TXD^{a}, where a = 0, is equal to one). The equations would seem to imply a break in sensitivity to TXD occurs at the value of 0.033 (.84 mm). This, however, is not true since there are no data between 0.033 (.84 mm) and 0.047 (1.19 mm), the first texture where significant increases in hydroplaning speed were observed. Obviously the change from insensitivity to sensitivity occurs between these values but the data present no way to define the break point, if there is such a thing, more closely. Observation of the way the two equations fit together as shown in Figure 13 indicates that the zone of change to a more sensitive condition may vary with water depth all the way across this range. However, the value of 0.033 (.84 mm) seems reasonably good as a break point for the water depths below 0.2 in. (5 mm), which are of more practical interest. For example, at a water depth of 0.1 in. (2.5 mm) the intersection of the equations occurs at about 0.027 (.69 mm) while at a water depth of 0.7 in. (18 mm) the intersection takes place at 0.054 in. (1.37 mm). This is reasonable since the influence of higher water depths would be expected to gradually overshadow low values of pavement texture. Another way to account for this slight discontinuity and the uncertainty about where the sensitivity break occurs is to put the equation in the following form:

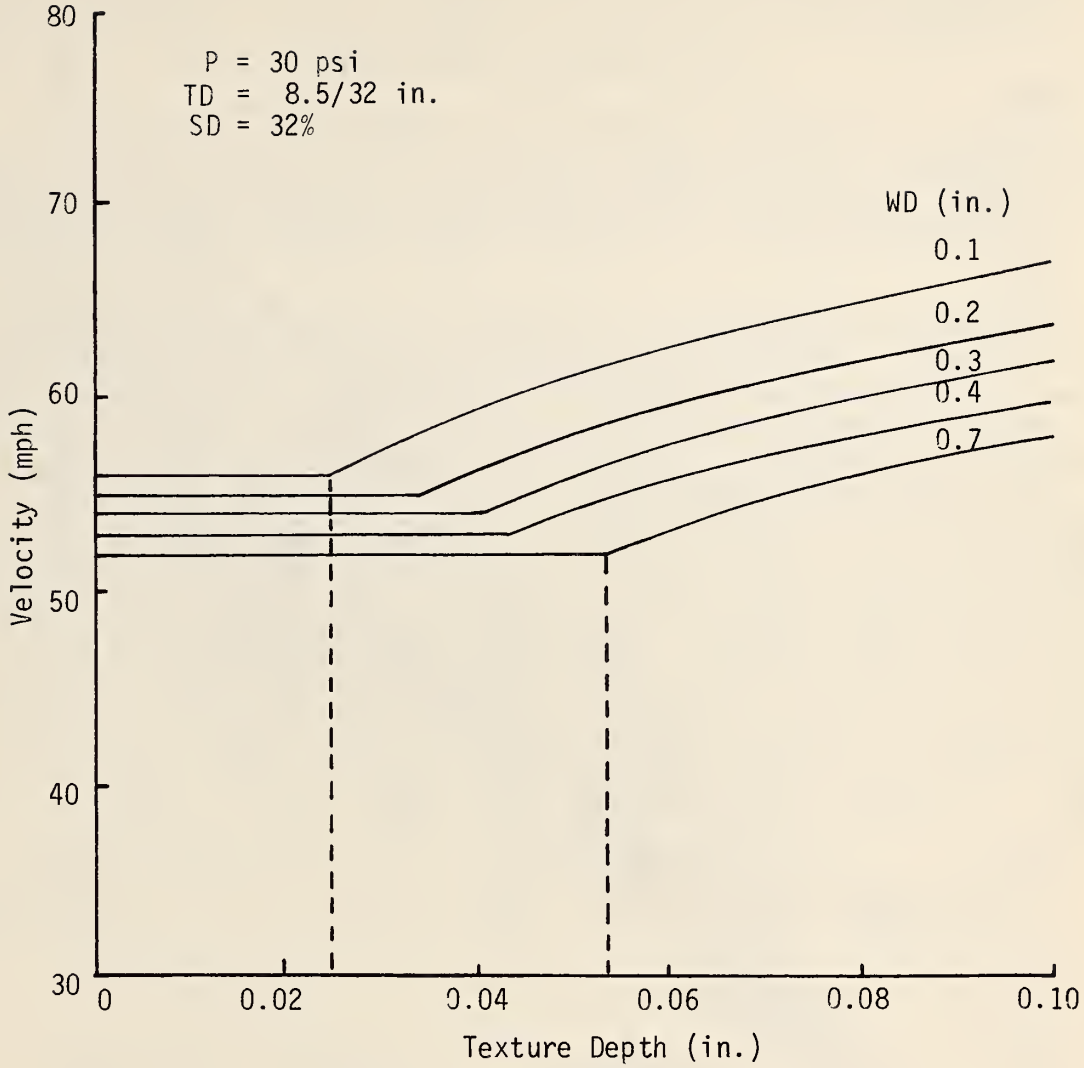
$$V = SD^{0.04}\ P^{0.3}\ (TD + 1)^{0.06}\ A \qquad (3)$$

Where A is the maximum of

$$\frac{10.409}{WD^{0.06}} + 3.507 \qquad \text{and} \qquad \left[\frac{28.952}{WD^{0.06}} - 7.817\right] TXD^{0.14}$$

In this case the maximum value of A would always be used, and the sensitivity break point would actually vary over the range previously indicated. The equation as shown is valid for the english units only. All constants would change if it were modified for appropriate metric units.

Figure 14 shows how the data are fit by Equation 3. Approximately 99% of the observations lie within plus or minus 5 mph (8 km/h) of the predicted value. The value of the calculated correlation coefficient is 0.85; not excellent, but reasonable.



Metric Conversion:

1 in. = 25.4 mm
1 mph = 1.609 km/h
1 psi = 6.894 kPa

Figure 13. Influence of texture depths and water depth on transition point.

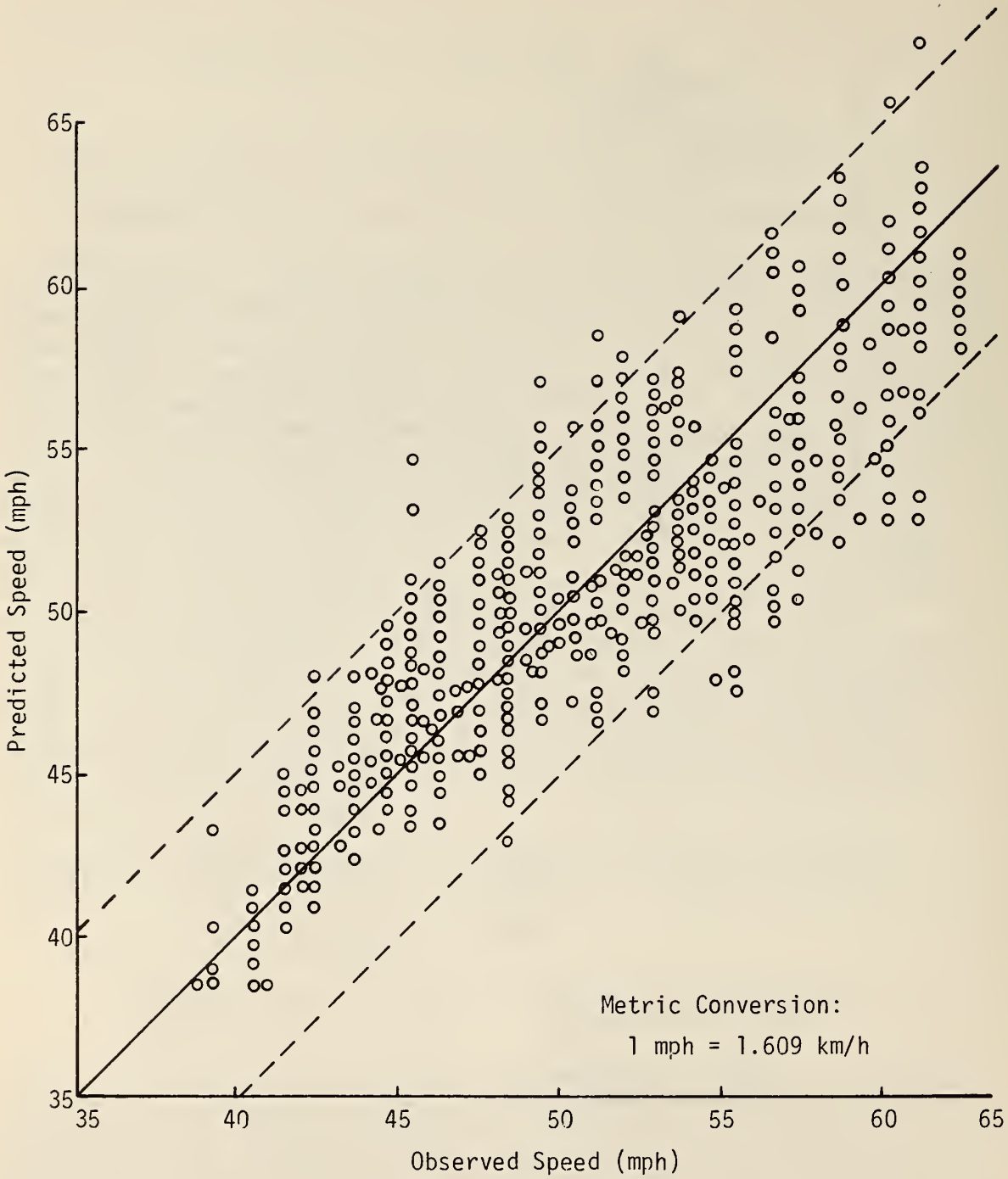


Figure 14. Data scatter for hydroplaning equation (data points at 10, 30 and 60% spin down).

Figure 15 shows the appropriate choice of A as a function of water depth and texture depth. Any combination of TXD and WD which is plotted below the boundary line would require the use of the first expression for A in Equation 3. Those combinations plotted above the boundary line would require the use of the second expression for A.

To illustrate the influence of the different parameters on the relationship between vehicle velocity and skid number, Figures 16 through 19 were plotted using Equation 3. Shown progressively are the influences of water depth (Figure 16), tread depth (Figure 17), tire pressure (Figure 18) and texture depth (Figure 19).

Limited direct evaluation of Equation 3 is provided by tests of two vehicles on flooded surfaces, performed as part of HPR-163. Table 2 shows a comparison of hydroplaning speeds predicted by Equation 3, column labeled "Computed Speed", with speeds at which it was observed that the vehicle was subject to negligible control forces, "Observed Speeds". The correspondence of these predicted and observed hydroplaning speeds is excellent considering the variations associated with full-scale vehicle tests, but is limited in the range of variables included. It was further noted in HPR-163 that the deterioration of control forces occurred rather gradually over a range of about 20 mph (32 km/h) shown in Figure 20, as full hydroplaning conditions were approached. What makes hydroplaning seem an abrupt occurrence is that on a straight tangent very little available friction is required to maintain direction, usually less than 0.1. However, the final deterioration of friction from 0.1 to 0 takes place in a range of only 1 or 2 mph (2 or 3 km/h) or can take place due to only a slight increase in water depth, or decrease in texture.

To depict the likely range in hydroplaning speed which may occur in the traffic mix along a highway, consider the following relatively extreme cases:

Vehicle 1	Vehicle 2
WD = 0.1 in. (2.5 mm)	WD = 0.1 in. (2.5 mm)
TD = 9/32 in. (7.14 mm)	TD = 0 in.
TXD = 0.15 in. (3.81 mm)	TXD = 0.01 in. (0.25 mm)
P = 36 psi (250 kPa)	P = 18 psi (125 kPa)

As shown in Figure 21 Vehicle 1 may travel up to 28 mph (45 km/h) faster than Vehicle 2 without risking hydroplaning. One of the major problems in speed zoning lies in this wide variation of vehicle speed. If the zone is set for the worst case, drivers of better equipped vehicles recognize it as too slow for them, assume it is too slow for everyone and, thus lose respect for speed zones.

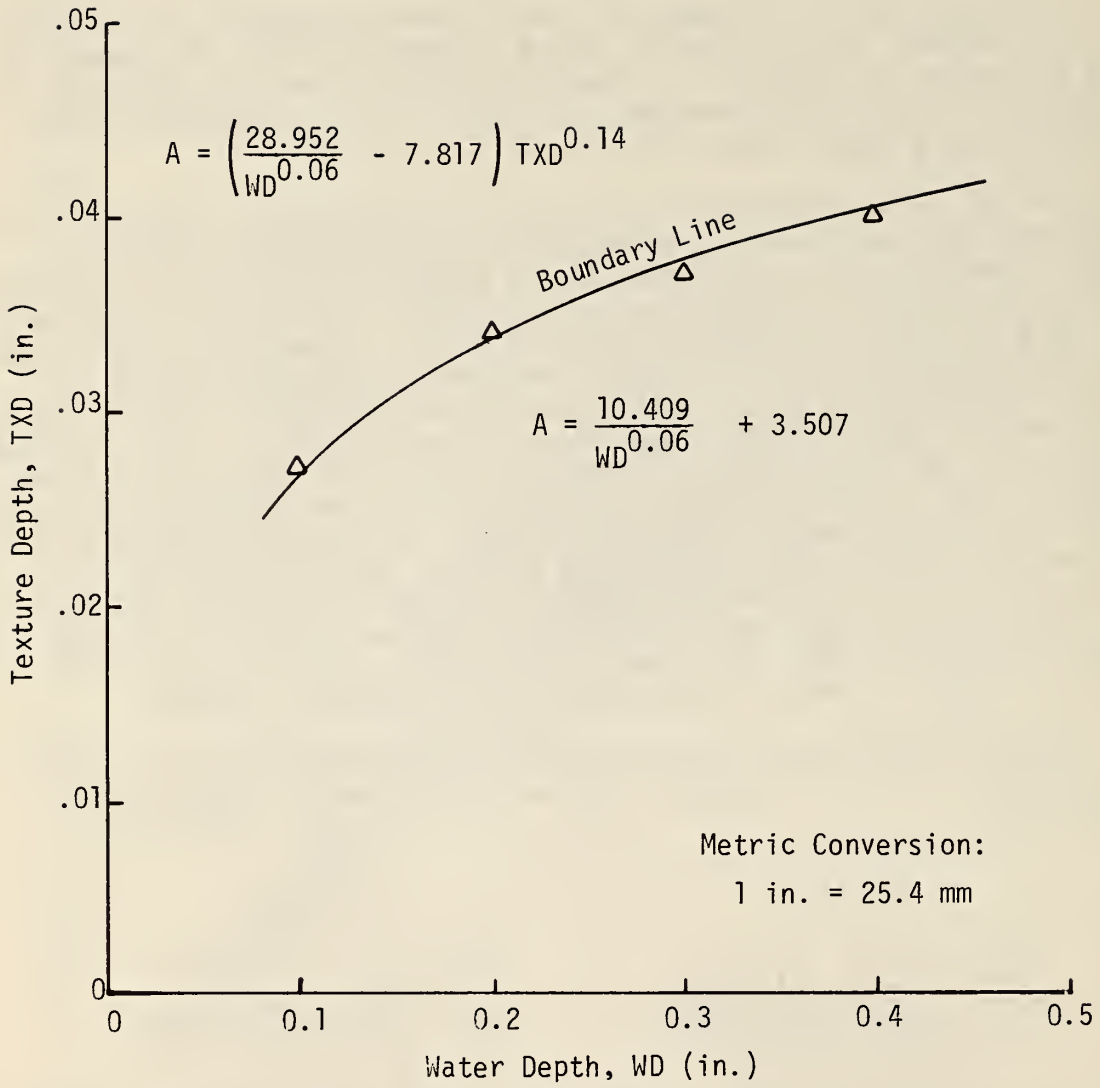


Figure 15. Choice of expression for A to be used in Equation 3.

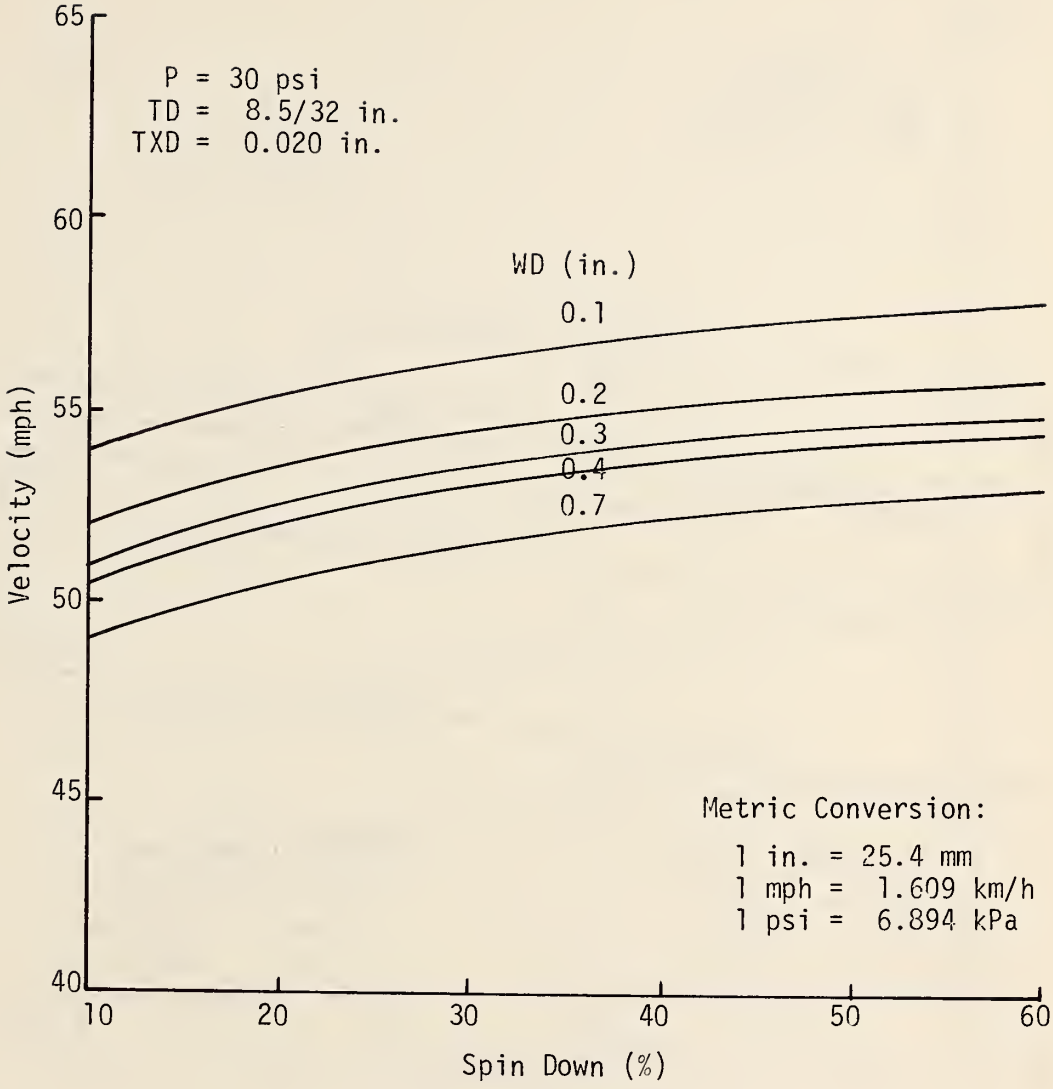


Figure 16. Influence of water depth on hydroplaning speed.

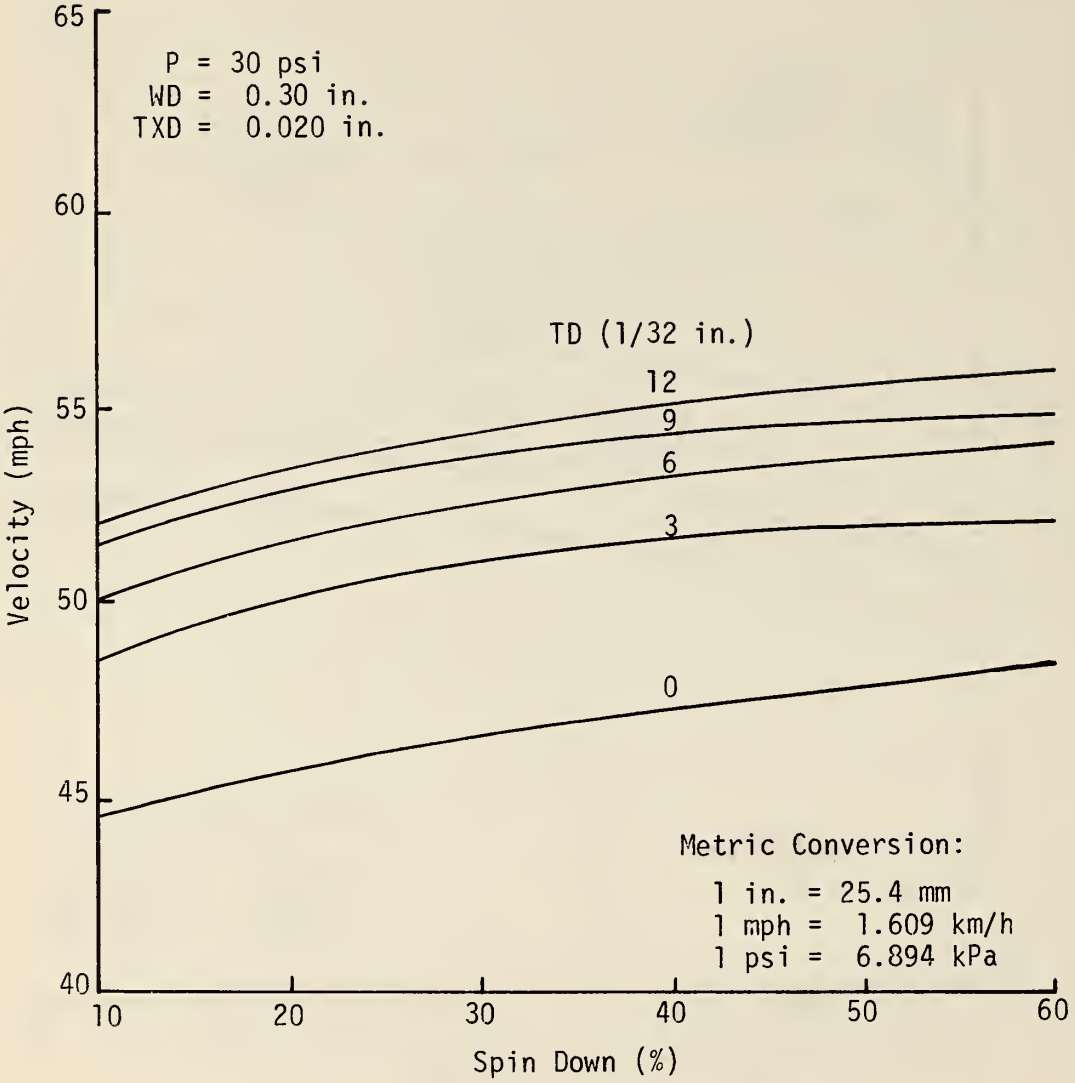


Figure 17. Influence of tread depth on hydroplaning speed.

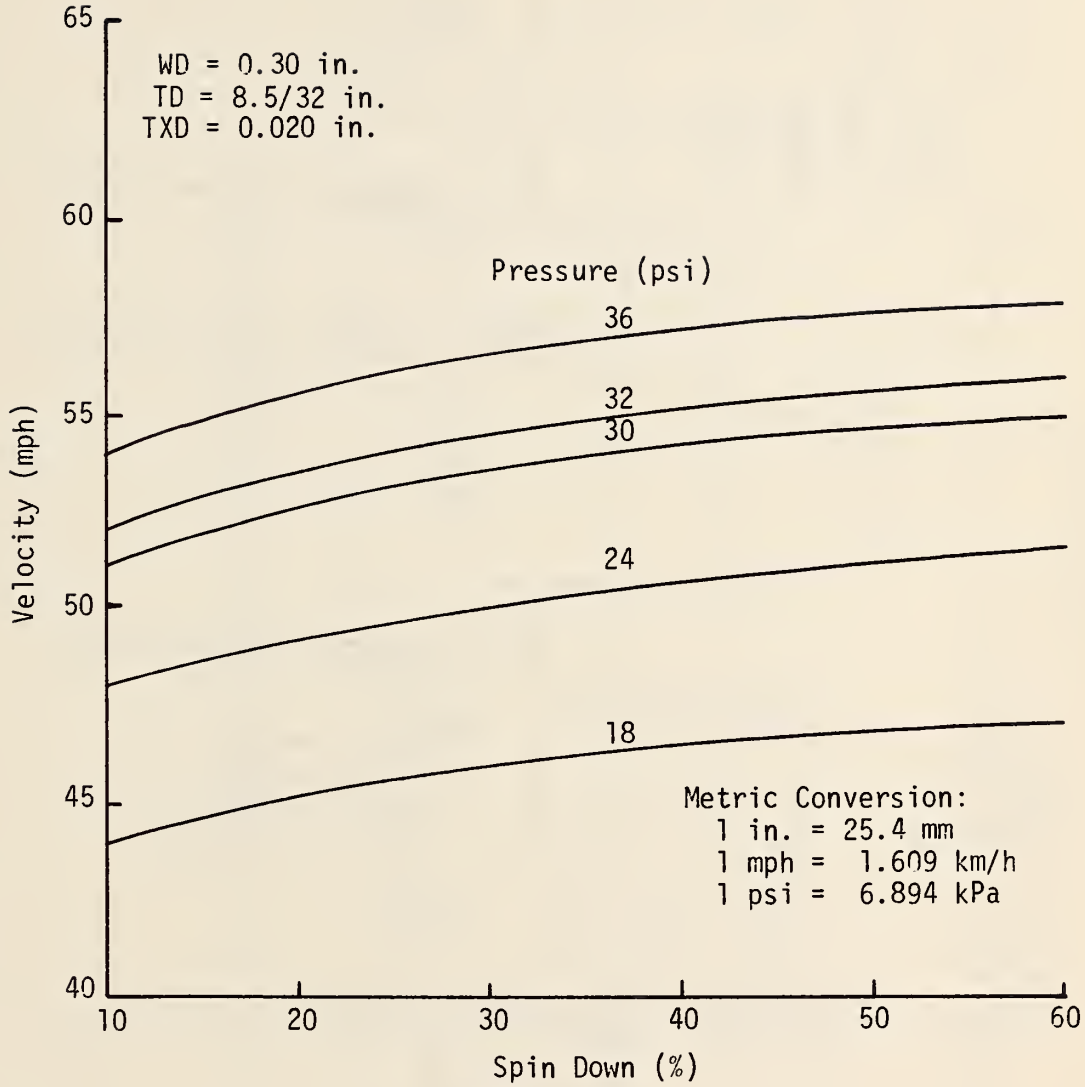


Figure 18. Influence of tire pressure on hydroplaning speed.

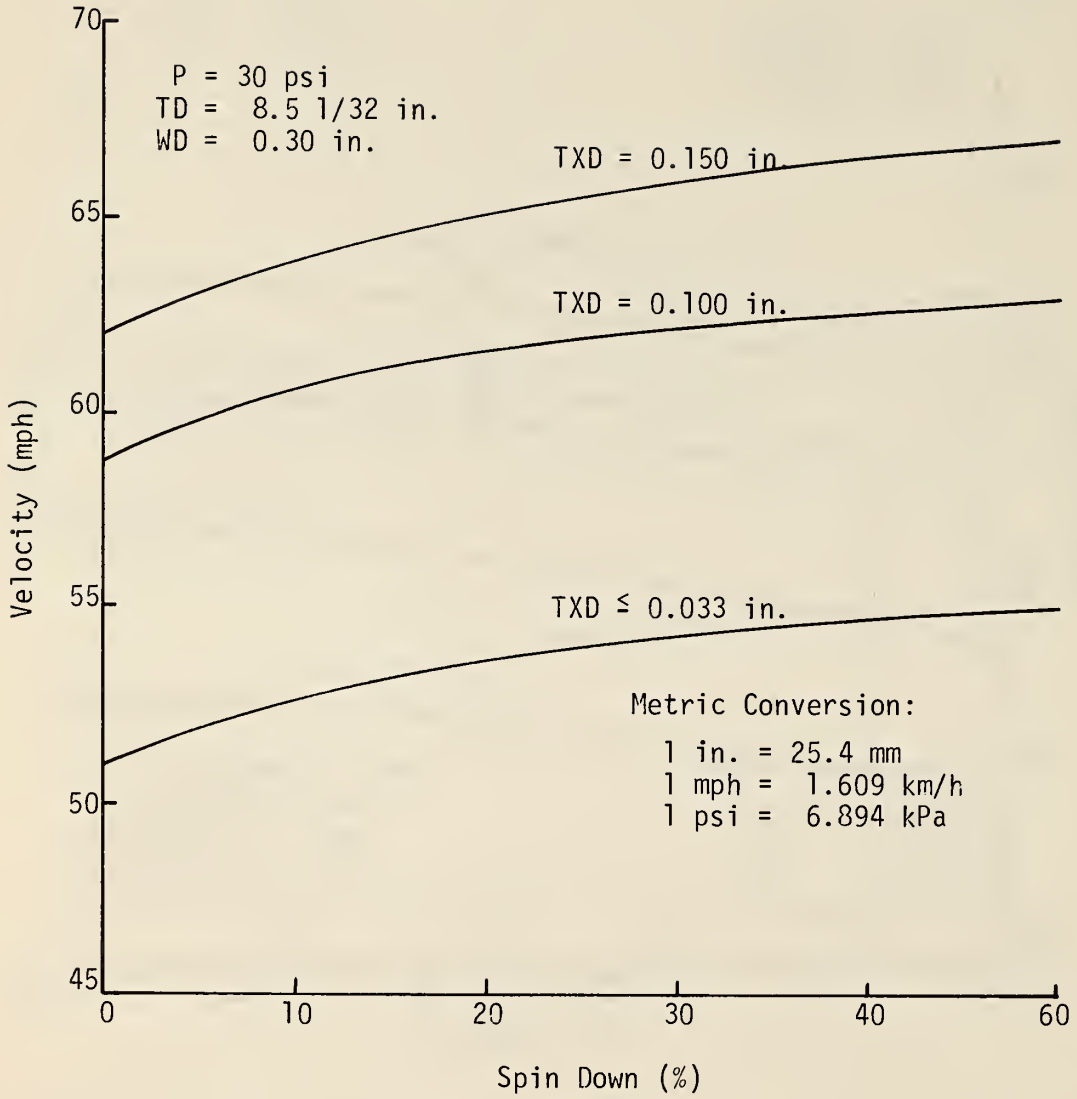


Figure 19. Influence of texture depth on hydroplaning speed.

Table 2. Computed and observed hydroplaning speeds.

Vehicle	Surface (No.)	Texture Depth (in.)	Average Water Depth (in.)	Tread Depth (32nds in.)	Tire Pressure (psi)	Computed Speed (mph)	Observed Speed (mph)
1964 Ford	2	0.014	0.10	8	26	53	53-57
1971 Ford	10	0.033	0.15	8	27	53	49-55
1971 VW	10	0.033	0.15	2	21	46	47-52

Surface Number 2 - Jennite Flush Seal

Surface Number 10 - Siliceous Rock Hot Mix

Metric Conversion: 1 in. = 25.4 mm
1/32 in. = 0.79 mm
1 psi = 6.894 kPa
1 mph = 1.609 km/h

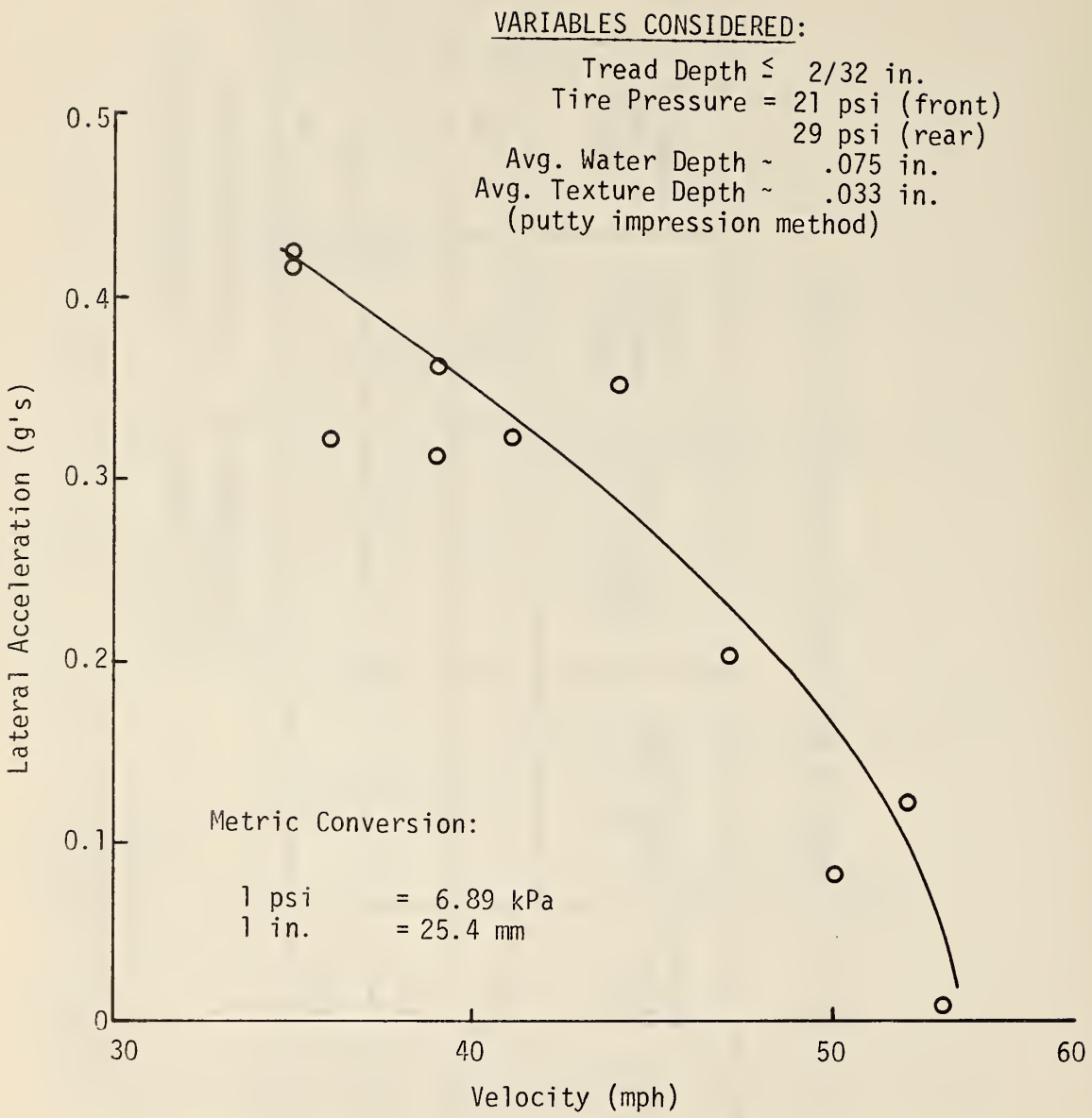


Figure 20. Deterioration of available friction with speed.

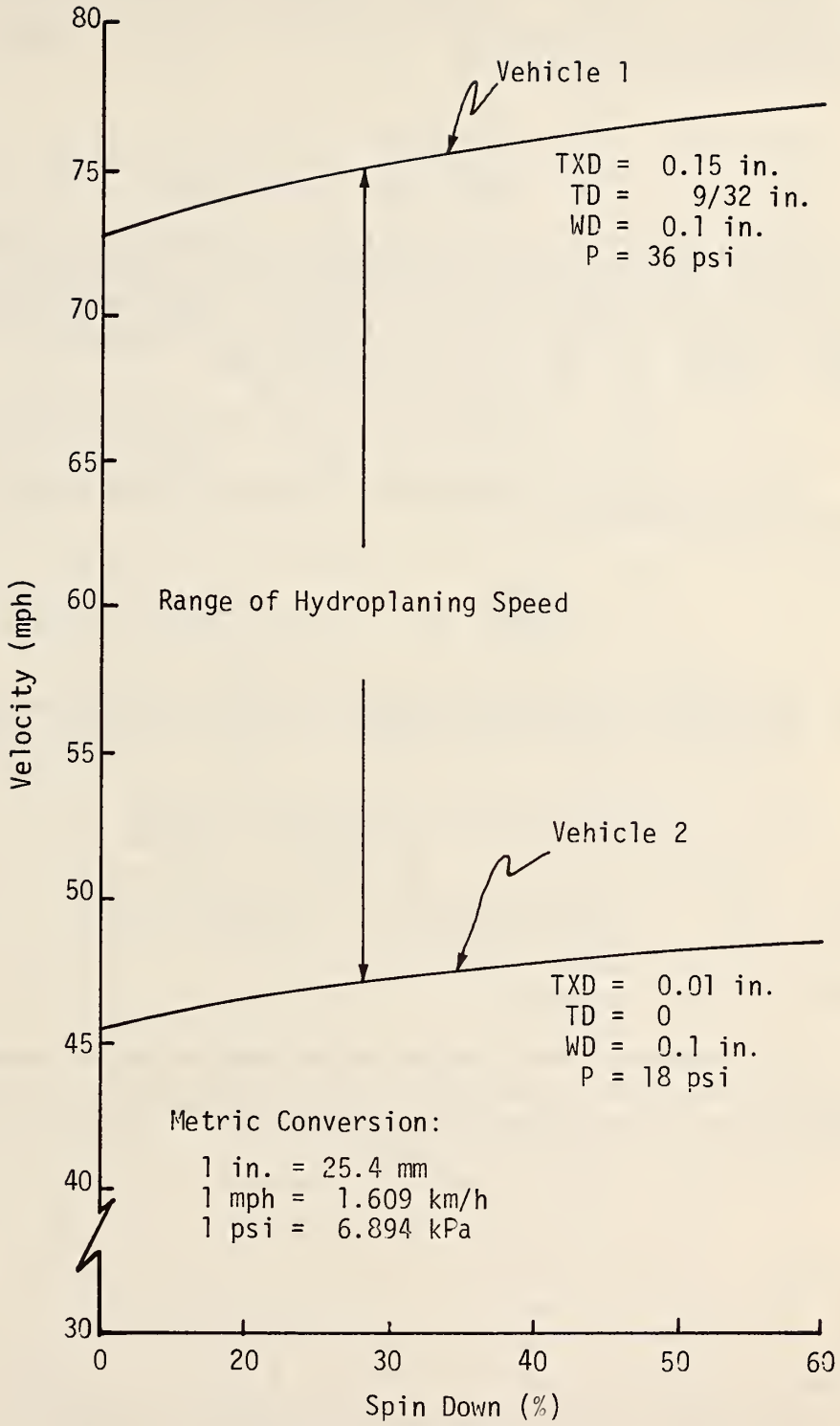


Figure 21. Possible range of hydroplaning speeds.

Now that an equation has been developed to predict the velocity for any degree of spin down for any situation where the other parameters may be specified, the point has been reached whereby a relationship between skid number and spin down may be developed. If, for example, the following equations were available:

$$SN = f_1(V, TXD, TD, WD, P) \tag{4}$$

and

$$V = f_2(SD, TXD, TD, WD, P), \tag{5}$$

it would be possible to substitute SD into Equation 5 giving V, then substituting V from Equation 5 into Equation 4 to predict SN. In this way a series of coincident values of SN and SD could be determined which could be used to ascertain the relationship between the two factors. Stated another way the relationship would be:

$$SN = f_1[f_2(SD, TXD, TD, WD, P)\ TXD, TD, WD, P]$$

Gallaway (7) has provided such an equation for random textured pavements in HPR Study 138.

$$SN = \frac{154}{V^{0.77}}\ [TD^{0.05} + \frac{4.71\ TXD^{0.09}}{(25.4\ WD + 2.5)^{0.09}}]* \tag{6}$$

This equation can be combined with Equation 3 by eliminating V to give SN as a function of spin down (SD) and the other pertinent variables, i.e.,

$$SN = \frac{154\ [TD^{0.05} + \frac{4.71\ TXD^{0.09}}{(25.4\ WD + 2.5)^{0.09}}]}{[SD^{0.4}\ p^{0.30}\ (TD + 1)^{0.06}\ A]^{0.77}} \tag{6a}$$

where A has been defined.

It is apparent that similar relationships can be developed from any of the SN equations given in the 138 and 141 reports. However, comparison of actual test values of SN might better illustrate the points that are pertinent to the current work.

---

*Equation developed from ASTM 14" tire @ 24 psi (165 kPa). (Intermediate to high random texture range, .020 to .150 in. (.508 to 3.81 mm)).

It might be assumed that at 100% spin down, values of SN would be the same on every pavement, that is, equal to that value of SN produced by hydrodynamic drag. However, both the data and the equations indicate this is not the case. For conditions which indicate 100% spin down by Equation 3, a rather wide range of SN values is indicated by the 138 data.

The best fit lines of SN vs. speed presented in Report 138-6 were used to calculate the correspondence of spin down at 2, 50 and 100% and SN. This resulted in the plot of SN vs. SD shown in Figure 22. Because of the heavy dependence on pavement texture, it can be concluded that full dynamic hydroplaning was not occurring when SN values were observed in the 138 study*, even though the conditions reported would indicate that hydroplaning should occur. At first glance this would seem to represent a serious inconsistency between the SN and SD studies. However, the problem is not inconsistency in the data taken under the two studies but in significant differences in the test conditions predominating when the two data groups were developed. The main difference is found in the water depth. In 147, spin down was observed in the presence of consistently positive water depths. In 138, SN values were observed in the presence of significantly lower average water depths which varied significantly about that average, the case more like actual highway conditions.

Figure 23 illustrates this variation with examples of water depth observations at ten different points under the rainmaker during the SN determinations for Study 138. Note the values of zero and negative water depths when in all examples the average water depth is positive. In this case periodic contact with the surface due to negative water depths on the significantly textured surfaces would drive the average value of SN up, as each SN value was determined over approximately 100 feet (30 m) of pavement length. Further, since even the higher water depths observed are rather low (less than .03 in. (0.8 mm)) there may not be sufficient time for the tire to hydroplane between negative water depth areas. One obvious point is that it is less likely that full hydroplaning will occur on a nonuniform surface where water depth varies significantly. Thus construction variations in the plane of the pavement surface may be working to prevent full hydroplaning, that is, *construction practices resulting in wavy and bumpy riding surfaces may be one of the best means of preventing hydroplaning*. Although highway engineers are not likely to allow contractors to produce these surfaces indiscriminately, the concept might be applied at certain critical points of poor pavement drainage such as the transition between a crowned section and a superelevated section. Care must be taken to prevent a surface so rough that it is disconcerting to drivers or roughness that is a hazard in itself. It should also be recognized that it is the driver's responsibility to reduce speed in keeping with extant environmental and roadway conditions. This responsibility can never be fully relieved by the roadway design, no matter how exacting.

*Except perhaps in the case of surfaces 2 and 9 where very low and convergent values of SN were observed.

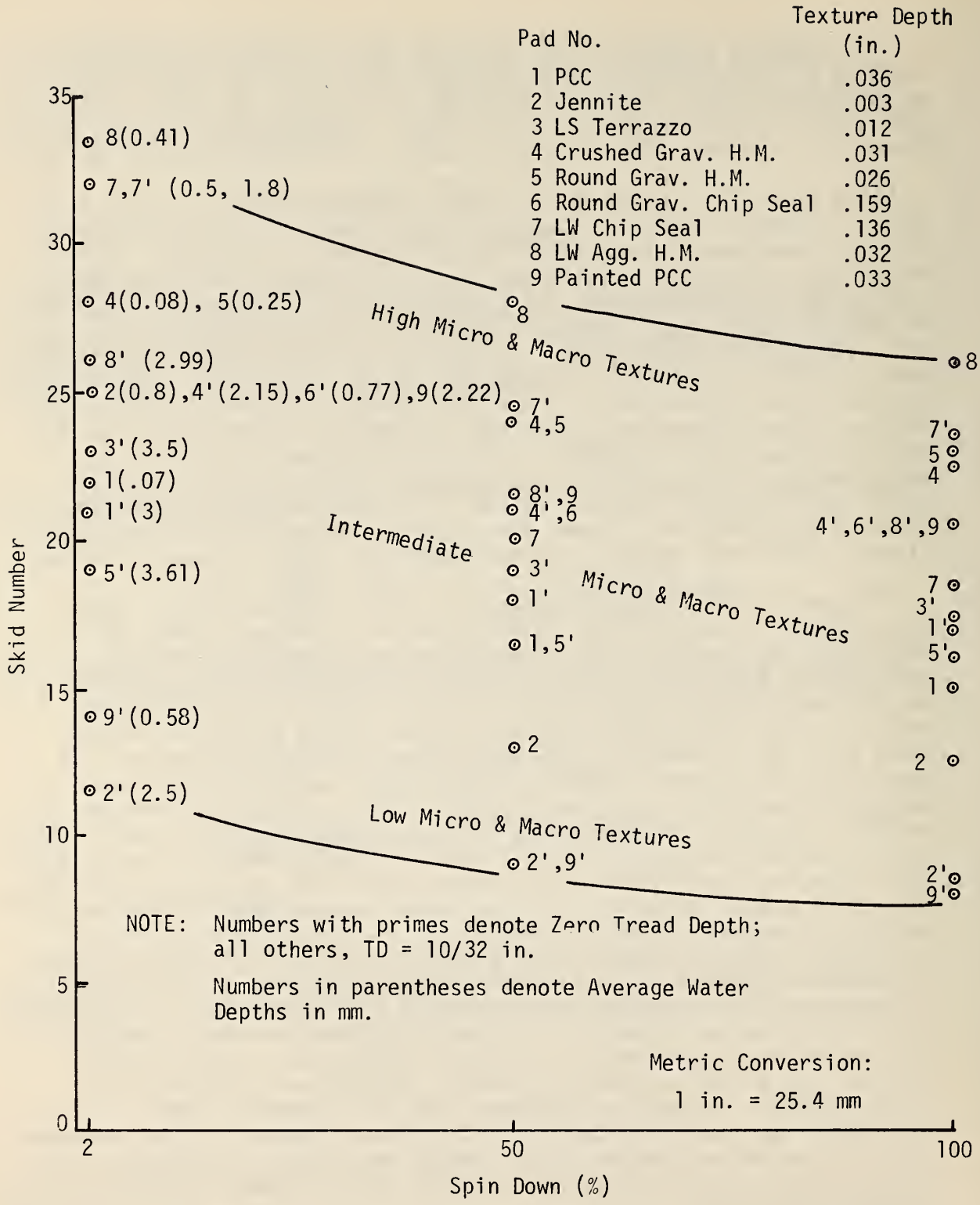


Figure 22. Skid numbers from 138 vs. spin down from 147.

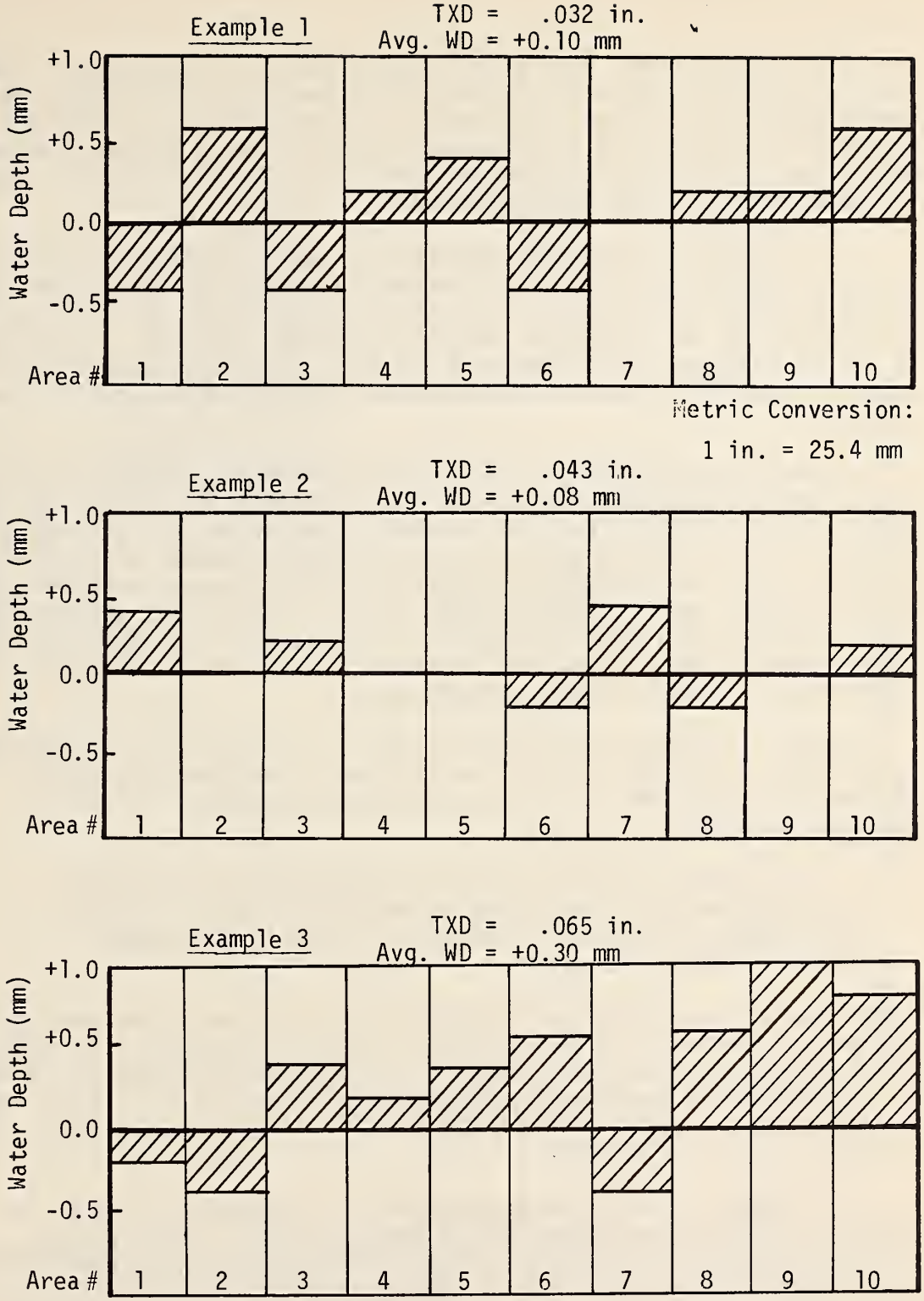


Figure 23. Variation of water depth during rainmaker SN tests.

It is obvious that this effect would be more pronounced on a highly textured pavement than on a low textured pavement. In the latter case, the rainfall has no place to go when it strikes, requiring a surface film type of flow to get to the depressed areas. For large textures, water will quickly flow off the asperities and travel in the channels between the aggregates to reach the lower areas.

In an effort to interpret the control forces available for vehicle maneuvers it would be valuable to estimate what part of the SN forces is due to friction between tire and pavement and what part is due to hydrodynamic drag as the tire moves across and through the water. An estimate of this hydrodynamic drag component may be developed by observing the skid numbers recorded at conditions which should obviously produce full hydroplaning.

Figure 24 shows average values of SN observed at 60 mph (97 km/h) for a number of different pavements. Again the relatively high SN is seen on the more highly textured pavements, indicating hydroplaning did not occur. On the low textured surfaces a lower boundary of about 7 SN is noticed. It seems probable that full hydroplaning was occurring at this time, thus the hydrodynamic drag (H.D.) accounted for approximately 7 SN. The real question is whether H.D. was larger in conditions where hydroplaning did not occur, thus contributing more to the SN values observed for the more highly textured pads.

Another estimate of hydrodynamic drag is available from Gengenbach (5) as shown in Figure 25. At 2 mm of water an apparent plateau is reached for hydrodynamic drag corresponding to about 22 lbs. (98N). Gengenbach states,

> "...The depth of immersion of the hydroplaning tire adjusts itself according to the velocity and water depth in such a way that the resistance of the water against displacement remains nearly constant."

If this force is interpreted in terms of force per unit tire width and applied to the tires studied under the 138 and 141 projects, it appears that hydrodynamic drag would amount to about three skid numbers. However, it is unclear from Gengenbach's presentation whether this is for a smooth or treaded tire. If it is for a smooth tire the drag may increase considerably for a treaded tire.

Examination of the SN data developed on patterned PCC may be of further value in interpreting the influence of small scale contact drainage patterns on SN values. Figure 26 shows that transverse textures give higher values of SN, even at values of predicted spin down up to 100%. This figure is developed from Equation 3 and from Equations 39 and 41 in Chapter IV.

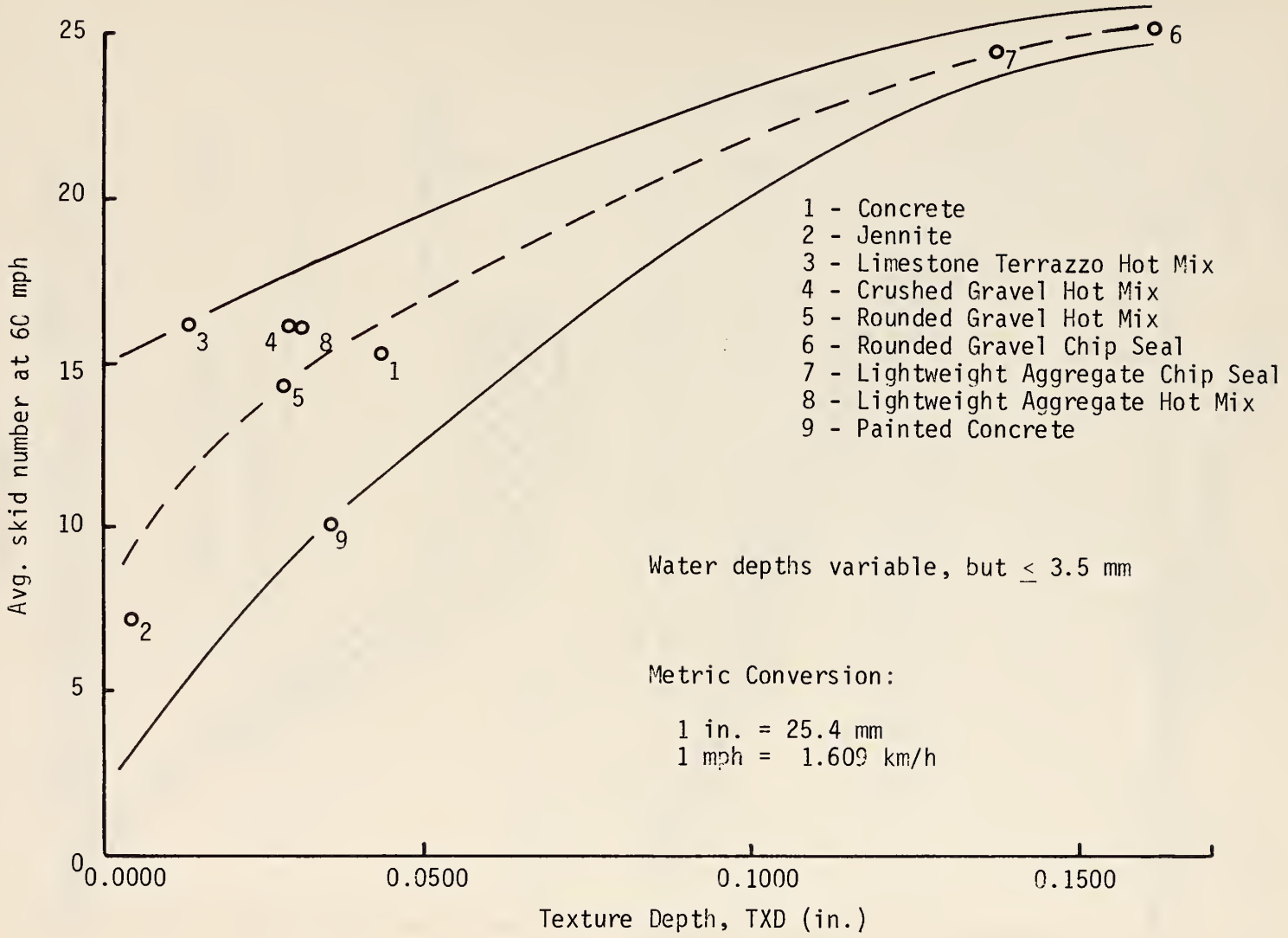


Figure 24. 60 mph skid numbers vs. texture depths.

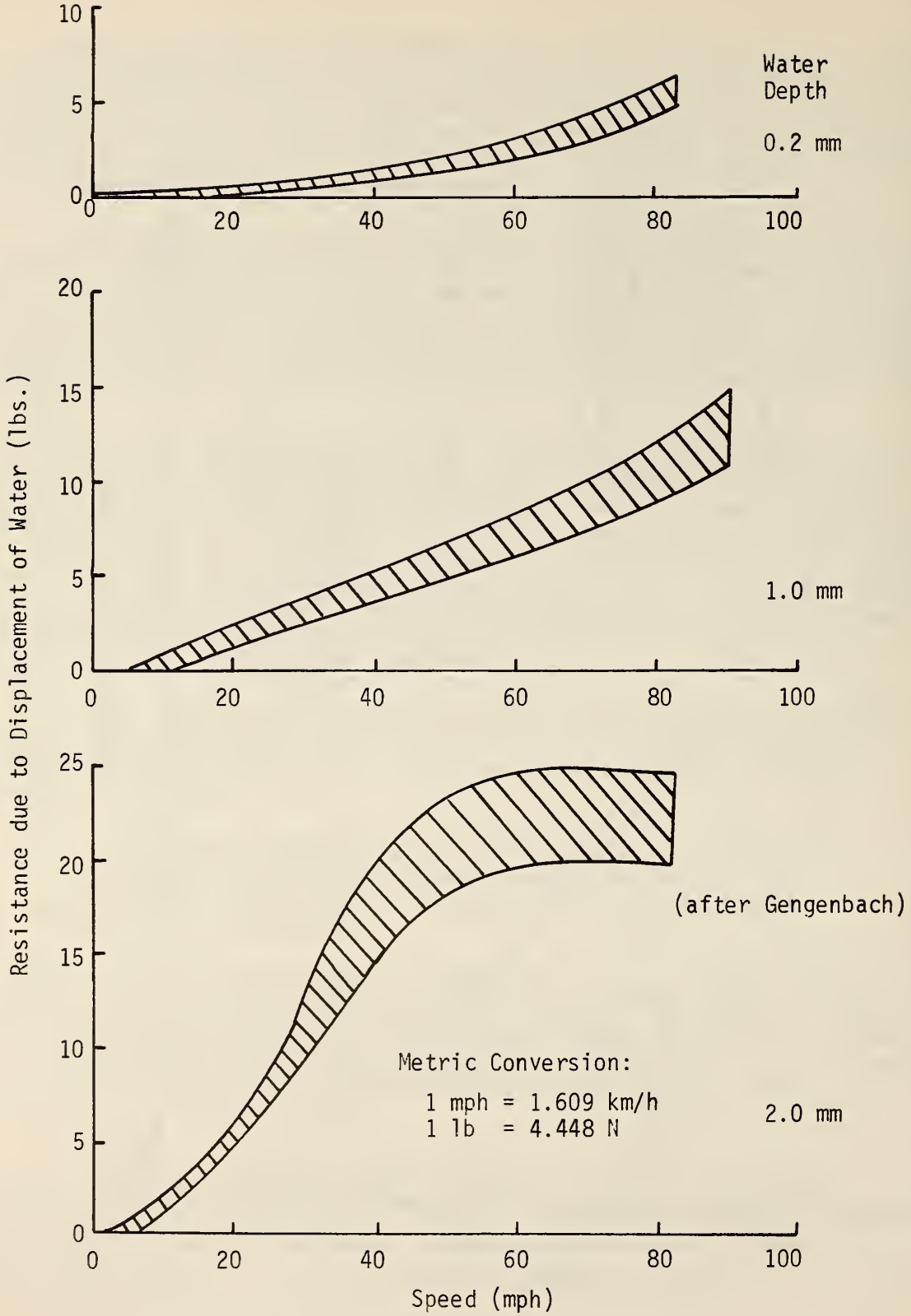


Figure 25. Resistance due to the displacement of water as a function of velocity for different water depths.

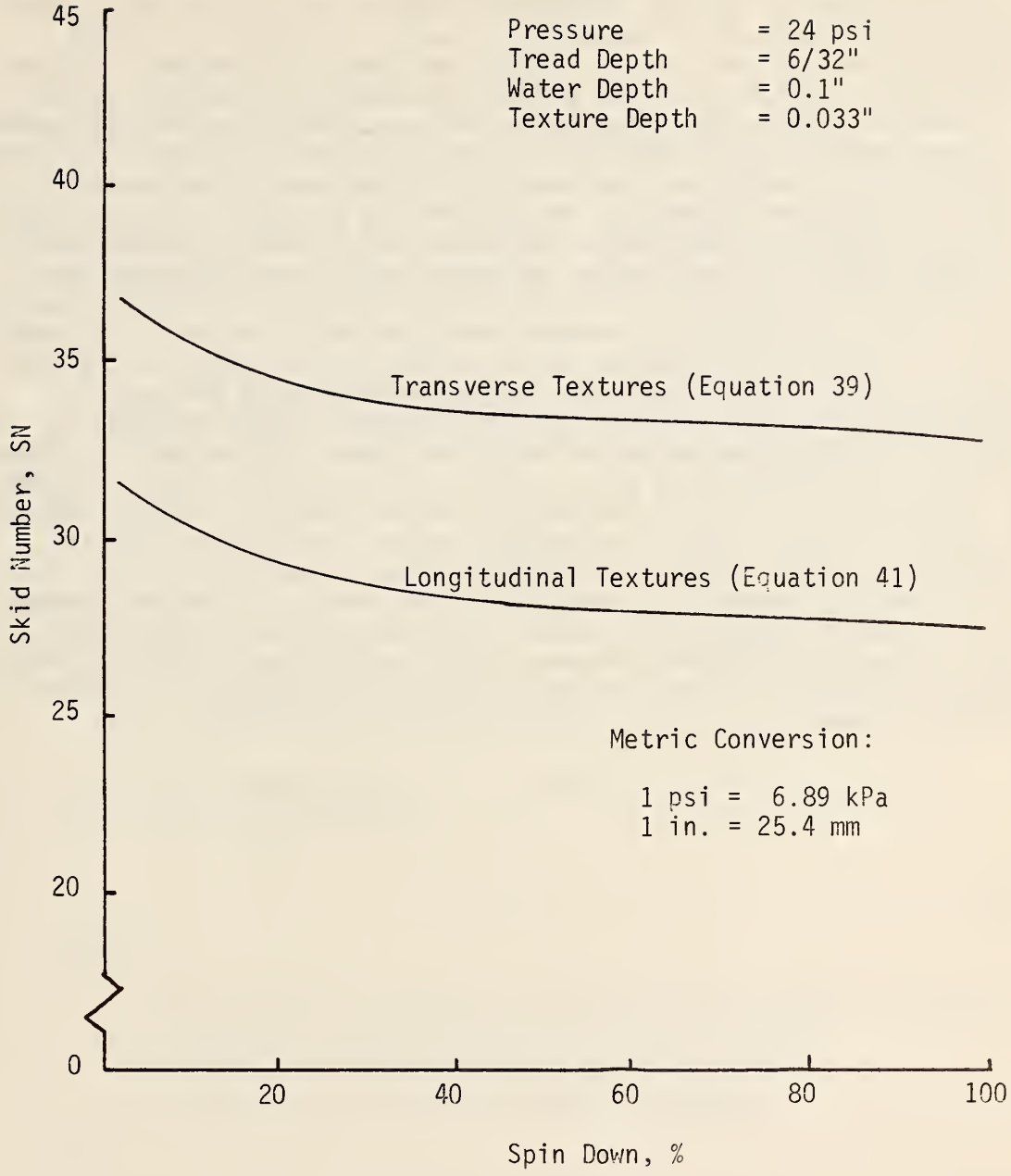


Figure 26. Effect of texture direction on skid numbers at calculated values of spin down.

When compared to the curves for random texture, Figure 26, the first conclusion might be that the transverse texture values were larger than values of the longitudinal texture, but such is not the case. Both orientations of texture covered roughly the same range between .020 and .070 in. (0.5 and 1.8 mm). The difference shown for a texture of .033 in. (.84 mm) is observed throughout this range. Why then this difference of approximately 25% in observed SN? The answer is threefold: (1) transverse textures, running in the direction of the cross slope, provide channels for better drainage of the pavement as a whole; (2) transverse pavement textures are more effective in draining the tire-pavement contact patch, just as cross grooves in the tire are more effective than circumferential grooves; and (3) transverse textures resist the forward motion of water necessary to produce the water wedge required for hydroplaning. Again the differences in the uniformity and depth of the water in the different studies that were discussed previously produce the apparent incongruity of relatively high SN values occurring simultaneously with the high values of wheel spin down. Although it is apparent that transverse texture gives larger values of skid number, it does not necessarily follow that transverse texture should be used throughout the highway system. This is because SN may not be indicative of the availability of the control force most needed in some parts of the system, i.e., cornering forces. It is apparent from studies of grooved pavement that longitudinal grooving increases cornering friction significantly, whereas transverse grooving increases only longitudinal braking friction. The same influence is likely to hold concerning longitudinal or transverse texturing.

CHAPTER III

PAVEMENT DRAINAGE

When rain falls on a sloped pavement surface, it forms a thin film that increases in thickness as it flows to the edge of the pavement as shown in Figure 27. Initially the water surface is below the top of the pavement macroasperities and the flow may be considered to be similar to porous media flow with a free surface. The water depth (WD) is then referred to as a negative water depth. As the drainage length (L) increases, the flow depth increases and eventually covers the pavement macroasperities. The flow may then be considered to be open channel flow.

If the water surface is below the top of pavement macroasperities, the flow may be assumed laminar, and Darcy's law will apply. Assuming the flow cross section is triangular as shown in Figure 28, the area of flow is

$$A = \left(\frac{WD_u}{TX}\right)^2 \tag{7}$$

where

TX = the maximum texture depth
WD_u = water depth above the bottom of the texture

If the area per unit width of pavement is assumed to be unity, $WD_u = TX$. Darcy's law states

$$q = kSA \tag{8}$$

where

q = discharge in cfs($(m^3/.028)/s$) per unit width of pavement
k = hydraulic conductivity in ft./s((m/.305)/s)
S = slope of free water surface
A = area of flow per unit width of pavement

and

$$q = kS\left(\frac{WD_u}{TX}\right)^2 \tag{9}$$

The equilibrium discharge rate from an elemental strip due to rainfall may be expressed as

$$q = \frac{IL}{43200} \tag{10}$$

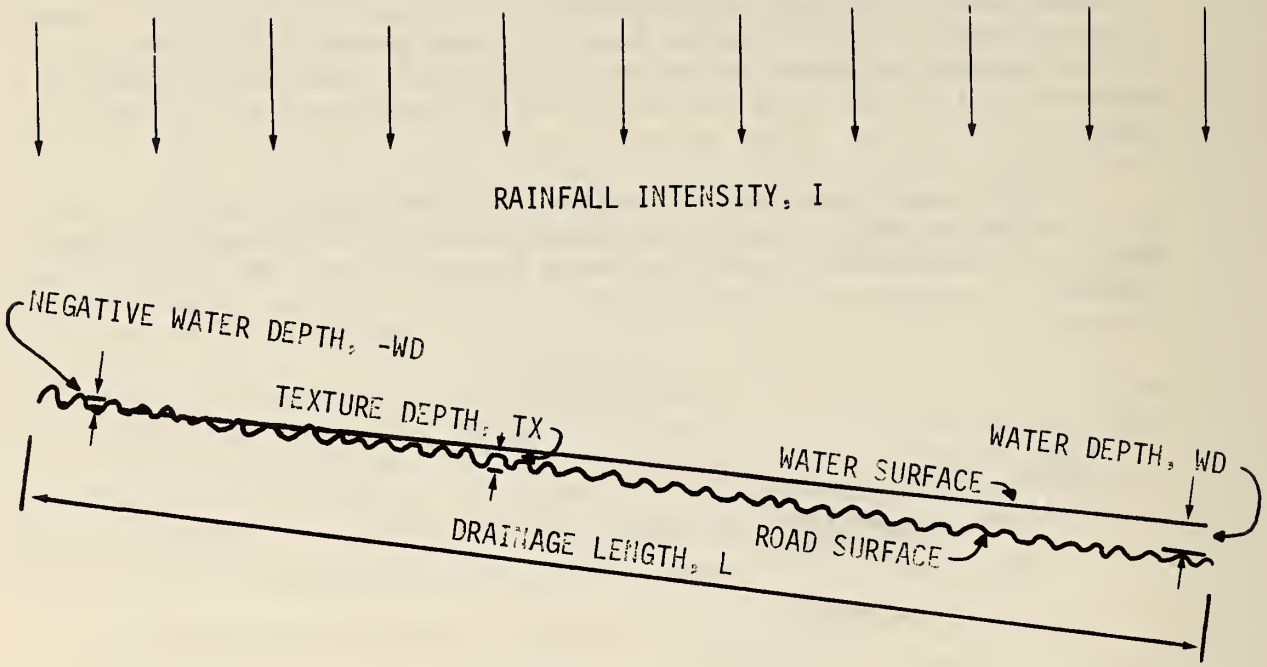
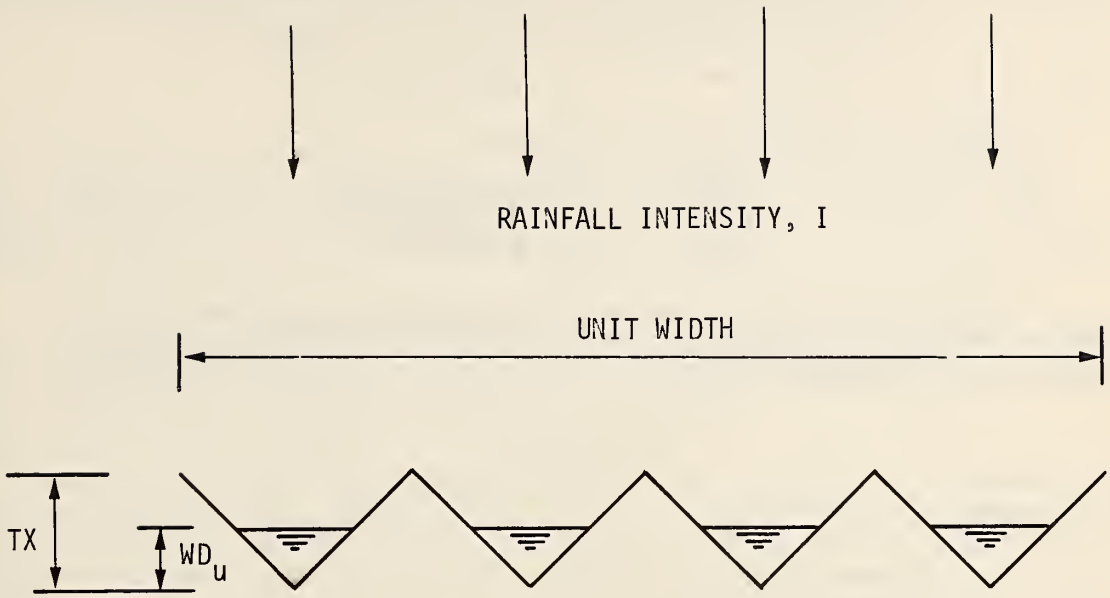


Figure 27. Diagrammatic representation of water film flow over a sloping road surface.



TX = MAXIMUM TEXTURE DEPTH

WD_u = WATER DEPTH ABOVE BOTTOM OF TEXTURE

Figure 28. Cross section of pavement macro asperities.

where

q = discharge rate in cfs ($(m^3/.028)/s$) per ft. (m/.305) width of pavement
I = rainfall intensity in in./hr. (25.4 mm/h), and
L = drainage length in ft. (m/.305).

Combining equations 9 and 10 gives

$$\frac{WD_u}{TX} = \sqrt{\frac{(IL)}{43200\ kS}} \tag{11}$$

If the water depth is above the top of the asperities, the flow may be considered open channel flow. Ackers (8) has suggested an equation of the general form:

$$V = K(WD)^{\alpha}S^{\beta} \tag{12}$$

where

V = mean velocity of flow
WD = depth of flow
S = slope and
K, α, β = constants for expressing the relationship between velocity, depth and slope.

The mean velocity for a unit width of open channel flow is:

$$V = q/WD \tag{13}$$

Combining equation 10 with equation 13, the expression for equilibrium discharge from an elemental strip due to rainfall may be expressed as:

$$V = \frac{IL}{43200\ WD} \tag{14}$$

and combining equation 12 with equation 14 gives

$$WD = (43200\ K)^{\frac{-1}{1+\alpha}}\ (IL)^{\frac{1}{1+\alpha}}\ \left(\frac{1}{S}\right)^{\frac{\beta}{1+\alpha}} \tag{15}$$

K', m and n may be substituted to simplify the equation as follows:

$$WD = K'(IL)^{m}\ \left(\frac{1}{S}\right)^{n} \tag{16}$$

it has been experimentally determined (8) that m and n are 1/3 for the case of laminar flow.

Paul G. Mayer (9) and M. T. Lighthill and G. B. Whitman (10) have independently determined the critical Froude number (N_F) to be 2 for open channels. The Froude number is herein defined as:

$$N_F = V/\sqrt{g\ WD} \qquad (17)$$

For the maximum slope tested (8%) in the Texas A&M water depth rainfall tests (11) on pad No. 3 with texture depth = 0.033 in. (0.08 mm), the maximum observed Froude number was 1.59, which occurred when the simulated rainfall intensity was 5.05 in./hr. (128 mm/h) and at L = 24 ft. (7.32 m).

The effect of raindrop impact on the flow has been disregarded in the above criteria for laminar flow. The impact of raindrops on the flow surface increases the turbulence of the flow and would cause turbulent flow at a lower critical value of N_F than specified above.

Paul G. Mayer (9) in his observations on roll waves and slug flows in inclined open channels, observed slug flows at lower Reynolds numbers when the flow was subjected to rainfall than when the flow was not subjected to rainfall. Rainfall thus does have a significant effect on increasing the turbulence of the flow. Slug flows result from instabilities which cause the transition from supercritical laminar to turbulent flow and the slug flows are characterized by a series of highly agitated surges separated by turbulent regions.

Gallaway, Schiller and Rose (11) reported on an extensive series of tests wherein nine different pavement surface types were subjected to various rainfall intensities for cross slopes covering the range from 1/2 to 8 percent.

The experimental sections under investigation were prototype surfaces produced by construction methods closely approximating full-scale contract work. These surfaces are generally described in Table 3 and Figure 29. The surfaces were four feet (1.22 m) wide and twenty-eight feet (8.53 m) in length representing a transverse segment cut out of a 2-lane highway. Pavement cross slope and texture were found to affect water depth significantly. The experimentally derived equation relating water depth to the variabiles studied is:

$$WD = 0.00338\ (TXD^{0.11}\ L^{0.43}\ I^{0.59}\ S^{-0.42}) - TXD \qquad (18)$$

where

WD = average water depth above top of texture in in. (mm/25.4)
TXD = average texture depth in in. (mm/25.4)
L = drainage path length in ft. (m/.305)
I = rainfall intensity in in./hr. (25.4 mm/h)
S = cross slope in ft./ft. (m/m)

Table 3. Descriptions of the surfaces studied for rainfall runoff. (after Gallaway, Schiller and Rose (11))

Surface Number	Surface Type	Aggregate Maximum Size (in.)	Texas Highway Department Specifications	Average Texture Depth** (in.)
1	Rounded Siliceous Gravel Portland Cement Concrete (Transverse drag)*	3/4	Class A Item 364	0.035
1A	Rounded Siliceous Gravel Portland Cement Concrete (Longitudinal drag)*	3/4	Class A Item 364	0.036
2	Clay Filled Tar Emulsion (Jennite) Seal	No Aggregate	--------	0.009
3	Crushed Limestone Aggregate Hot Mix Asphalt Concrete (Terrazzo Finish)	1/2	Type D Item 340	0.003
4	Crushed Siliceous Gravel Hot Mix Asphalt Concrete	1/4	Type F Item 340	0.019
5	Rounded Siliceous Gravel Hot Mix Asphalt Concrete	5/8	Type C Item 340	0.039
6	Rounded Siliceous Gravel Surface Treatment (Chip Seal)	1/2	Grade 4 Item 320	0.141
6	Rounded Siliceous Gravel Surface Treatment (Chip Seal)	1/2	Grade 4 Item 320	0.141
7	Synthetic Light-weight Aggregate Surface Treat-ment (Chip Seal)	1/2	Grade 4 Item 320	0.164
8	Synthetic Light-weight Aggregate Hot Mix Asphalt Concrete	1/2	Type L Sp. Item 2103	0.020

*With respect to direction of vehicular travel
**Obtained by Putty Impression Method

Metric Conversion:
1 in. = 25.4 mm

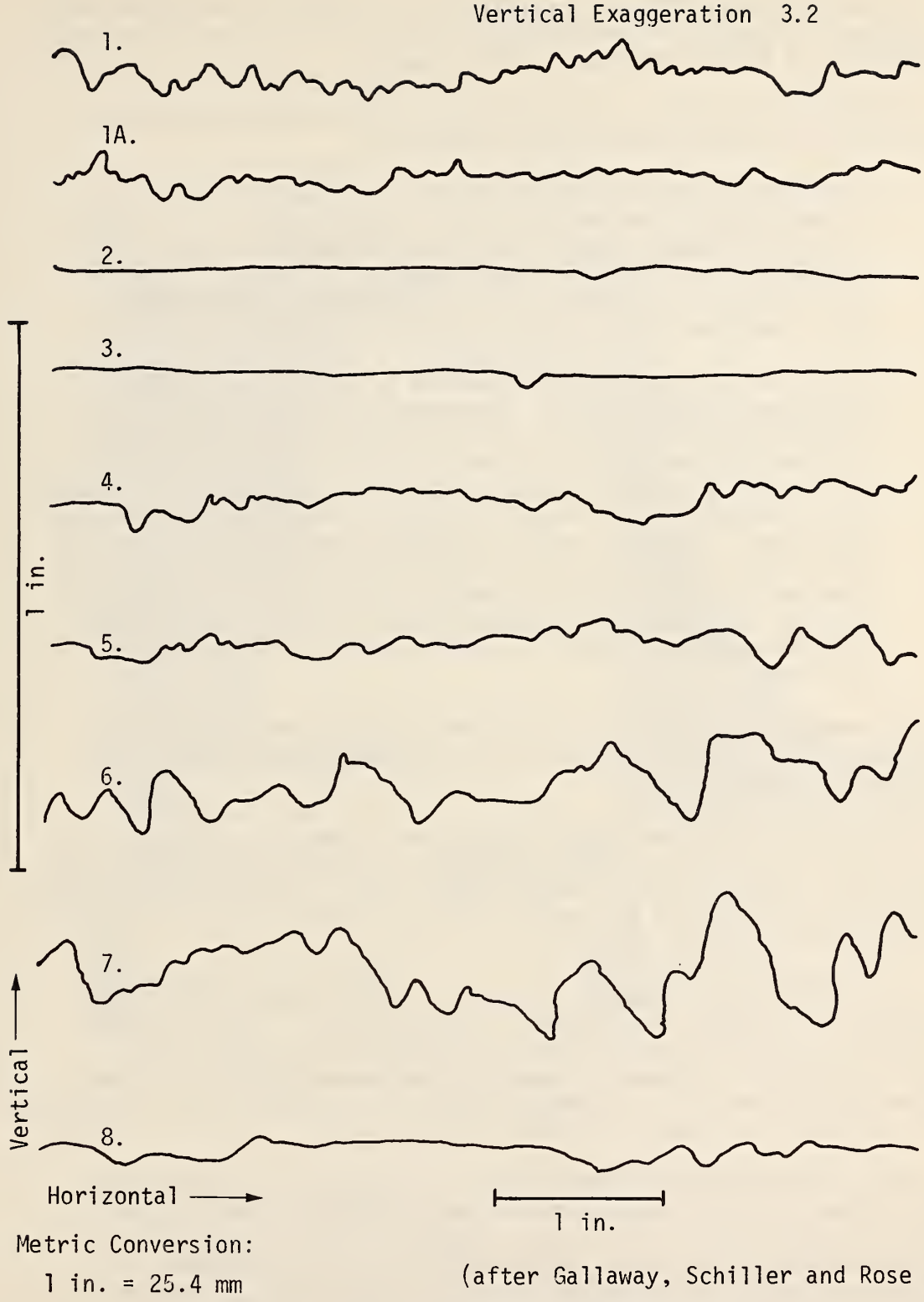


Figure 29. Textural profiles of the surfaces.

A better appreciation of the relationship among the variables may be obtained by studying Figure 30 while taking into account the following sequence of events relating to this figure.

A rainfall of uniform intensity falling on a given pavement will result in the following events (see Figure 30).

1. Initially a certain amount of water is required to fill the interstices of the surface before runoff occurs. This amount is referred to as "depression storage" and is measured in volume per unit area or average depth in inches. It depends on the initial wetness of the surface, degree of surface texture, deformations in the surface and cross slope.

2. After the amount of water required for depression storage is satisfied, runoff begins. The runoff rate increases to an equilibrium value, and for an impermeable surface, this rate is equal to the rainfall intensity. It is during this interval that the amount of water detained on the surface increases to a maximum value. The thin sheet of water on the surface at the time of constant runoff, excluding that required for depression storage, is called "surface detention." It has the same units as depression storage and can also be expressed as a value at a point or an average over an area. Surface detention depends primarily on the cross slope and rainfall intensity.

The momentary increase in runoff which occurs immediately after the cessation of rainfall is due to loss of turbulence caused by impact of the raindrops.

It is also recognized that, in addition to reduced traction, surface detention water contributes to splash and depression storage water produces spray effects after runoff ceases. This condition can persist quite long and contribute to reduce friction and cause poor visibility via windshield splatter.

Data collected on these surfaces for rainfall rates which vary from about 0.3 to about 5.5 in./hr. (8 to 140 mm/h) are presented in Ref. 11 and relationships between variables are illustrated by Figures 31 and 32. The general equation for water depth was derived from the data in Ref. 11. A summary analysis presenting the interaction of the variables is shown in Table 4.

These data should be helpful in the interpretation of the recommended values for texture and cross slope to minimize hydroplaning. In these studies (11) it was further suggested that research be conducted to determine the relative influences of ambient temperature, wind velocity and relative humidity on the drying rates of various pavement types following cessation of rain. Drying rates bear a direct relationship with the time a pavement surface is wet after

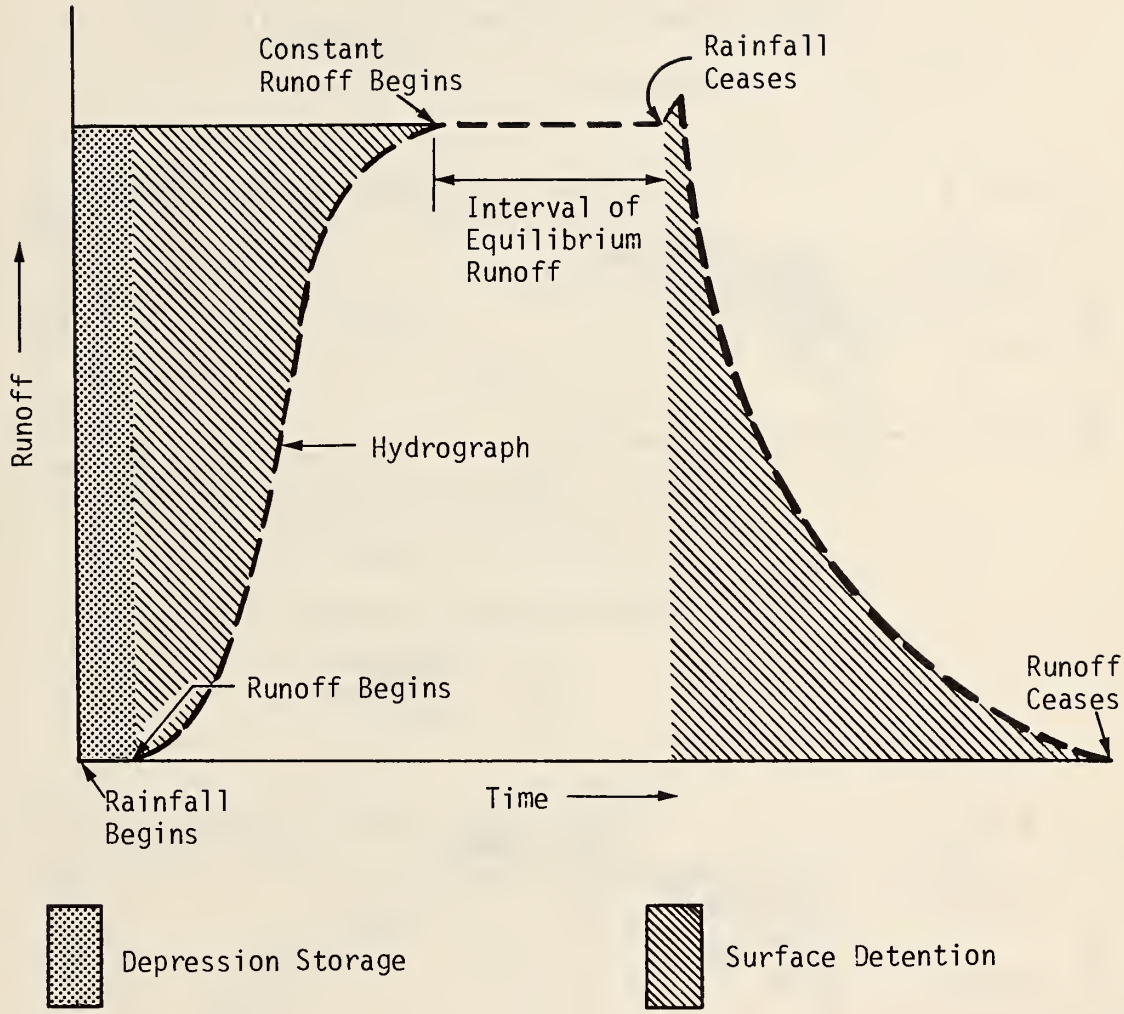


Figure 30. Rainfall runoff hydrograph.

(after Gallaway, Schiller and Rose (11), 1971)

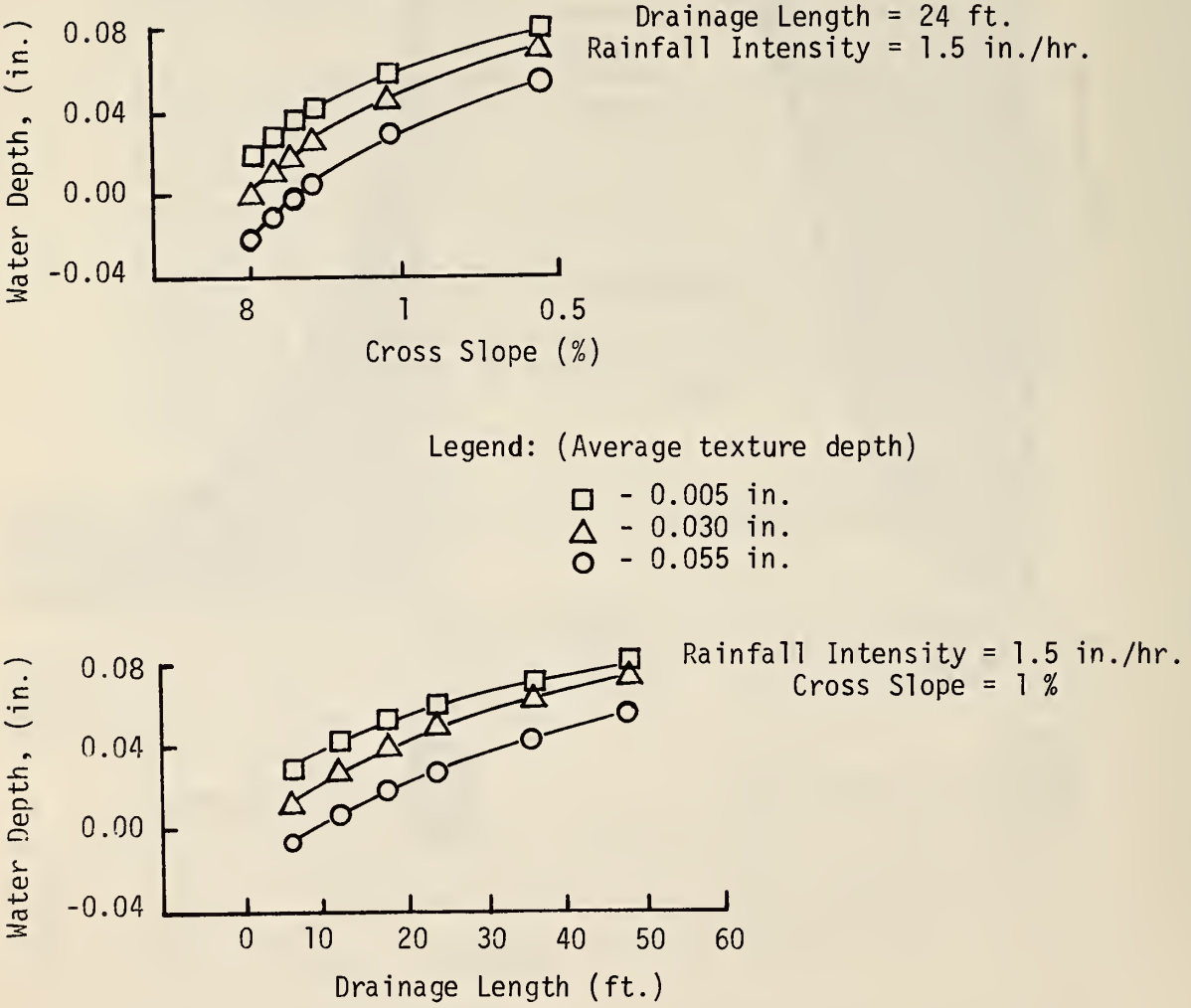


Figure 31. Plot of water depths vs. variables for combined surfaces (points calculated from Equation 18).

(continued next page)

(after Gallaway, Schiller and Rose (11), 1971.)

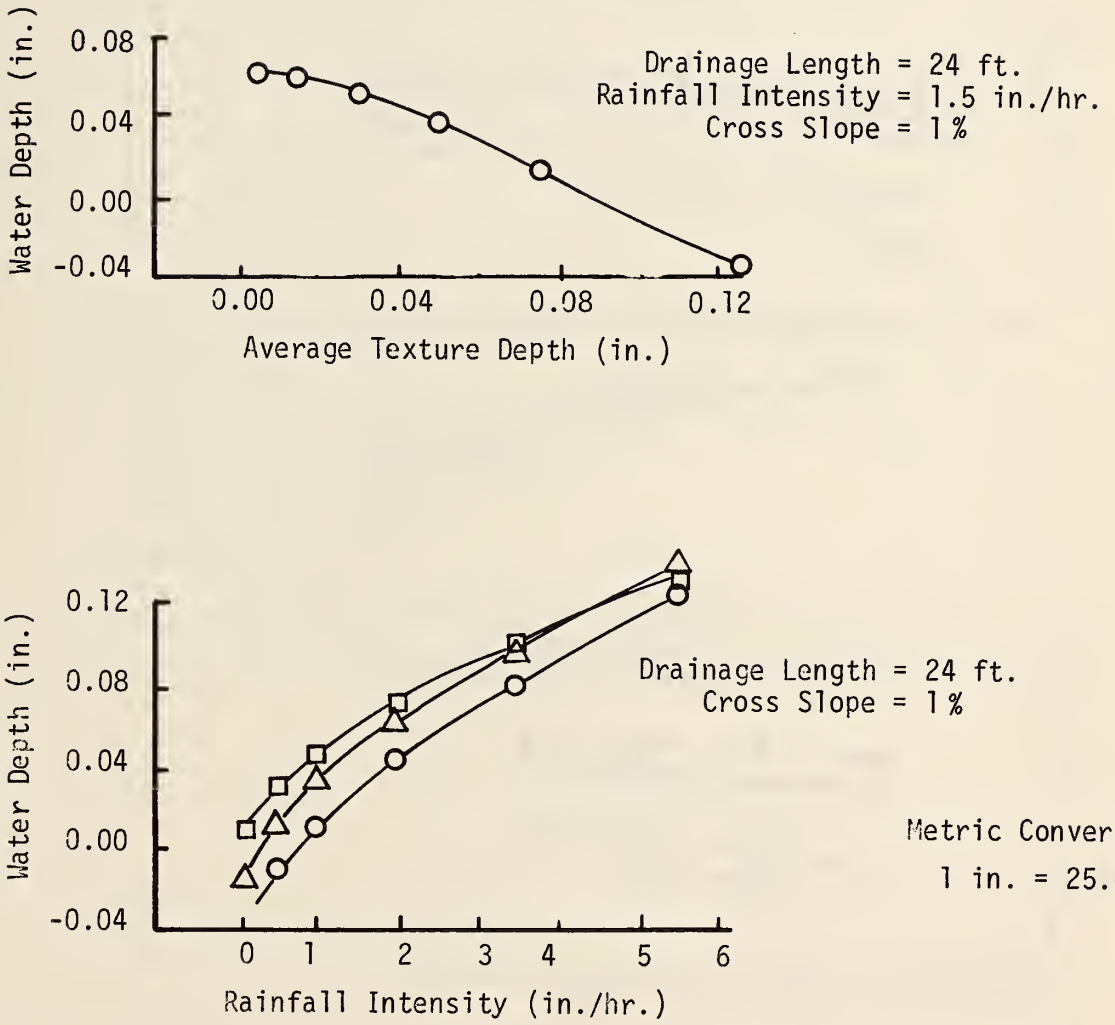
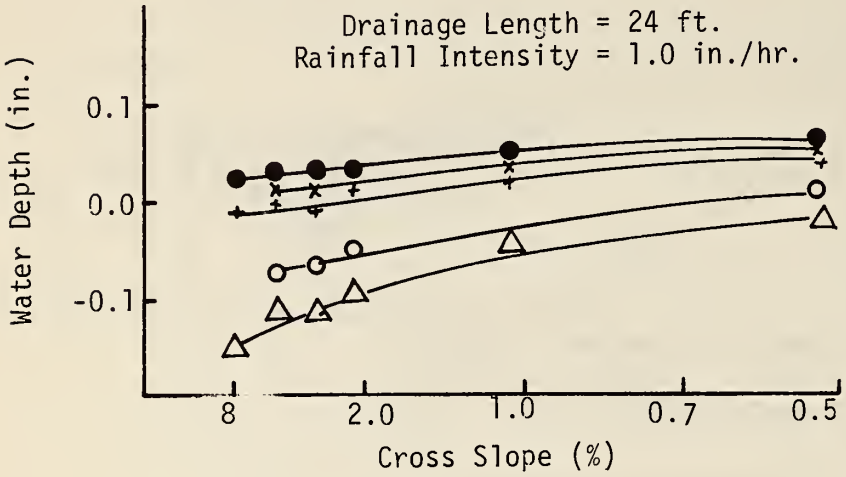


Figure 31. Plot of water depths vs. variables for combined surfaces (points calculated from Equation 18).

SURFACE LEGEND		SURFACE TYPE	TEXTURE DEPTH, in.
1	x	Portland Cement Conc. (transverse drag)	0.035
1A	+	Portland Cement Conc. (long. drag)	0.036
3	●	Terra 330 Finish Hot Mix Asph. Conc.	0.003
6	○	Rounded Gravel Chip Seal	0.141
7	△	Lightweight Aggregate Chip Seal	0.164



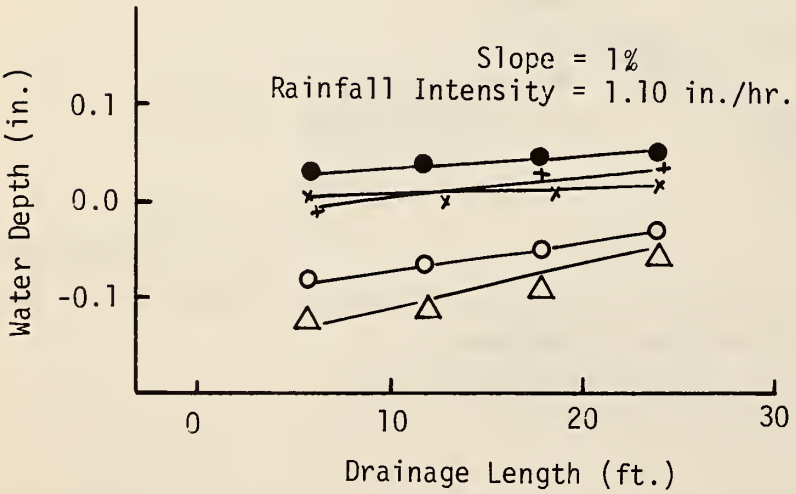
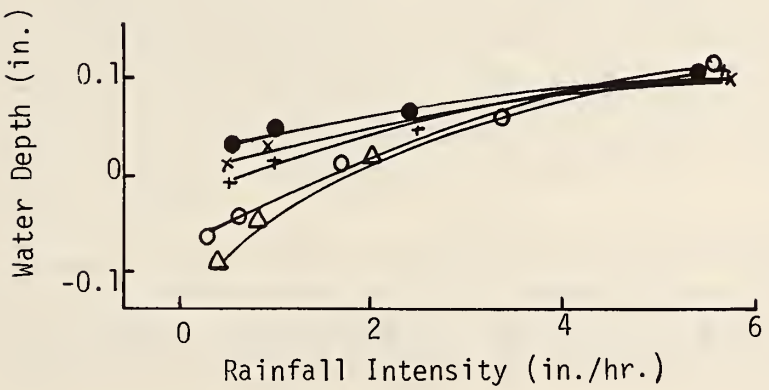


Figure 32. Plot of water depths vs. variables of selected individual surfaces (observed or average data points).
(continued next page)

SURFACE LEGEND	SURFACE TYPE	TEXTURE DEPTH, in.
×	Portland Cement Conc. (transverse drag)	0.035
+	Portland Cement Conc. (longitud. drag)	0.036
●	Terra 330 Finish Hot Mix Asph. Conc.	0.003
O	Rounded Gravel Chip Seal	0.141
△	Lightweight Aggregate Chip Seal	0.164



Metric Conversion:

1 in. = 25.4 mm
1 ft. = .305 m

Figure 32. Plot of water depths vs. variables of selected individual surfaces (observed or average data points).

Table 4. Tabular representation of the relative effects of cross slope, texture, drainage path length and rainfall intensity on water depth.

Constants	Variable Cross Slope (in./ft.-%)	Resultant Water Depth (in.)	Percent Decrease
Texture 0.03 in.	1/16-0.5%	0.074	
	1/8 - 1%	0.048	35
Length 24 ft.	1/4 - 2%	0.028	62
	3/8 - 3%	0.021	72
Intensity 1.5 in./hr.	1/2 - 4%	0.013	82
	1 - 8%	0.002	97

Constants	Variable Drainage Length (ft.)	Resultant Water Depth (in.)	Percent Increase
Intensity 1.5 in./hr.	6	0.013	
	12	0.028	115
Cross Slope 1/8*- 1%	18	0.039	200
	24	0.048	269
Texture 0.03 in.	36	0.062	377
	48	0.074	469

Constants	Variable Texture (in.)	Resultant Water Depth (in.)	Percent Decrease
Length 24 ft.	0.005	0.059	
	0.015	0.057	3
Intensity 1.5 in./hr.	0.030	0.048	19
	0.050	0.038	36
Cross Slope 1/8*- 1%	0.075	0.011	81
	0.125	-0.034	158

Constants	Variable Rainfall Intensity (in./hr.)	Resultant Water Depth (in.)	Percent Increase
Texture 0.03 in.	0.5	0.011	
	1.0	0.031	182
Length 24 ft.	2.0	0.062	464
	3.5	0.098	791
Cross Slope 1/8*- 1%	5.5	0.138	1154

*in./ft.

Metric Conversion:

1 in. = 25.4 mm
1 ft. = .305 m

rain stops. For a selected cross slope a fine texture pavement would be expected to drain and dry more quickly than a pavement with a coarse texture. However, this advantage may have a very small effect on the overall wet weather performance of such a surface.

Other studies of water depth on road surfaces have been conducted by the Road Research Laboratory (RRL) (12) and by the Goodyear Tire and Rubber Company (2). Analyses of the experimental basis of the available equations reveal that only the TTI equation is appropriate for the prediction of water depths close to the vertical position of the pavement asperities for a range of different pavement textures. The RRL equation was based on observations on two pavements exhibiting texture depths of 0.072 in. (1.8 mm) and 0.095 in. (2.4 mm) respectively. The equation presented in NCHRP Report 1-14 was based on the Goodyear data which included only one pavement, a low texture concrete (0.010 in. (.25 mm)). Discussion with researchers at Goodyear and at RRL revealed further that the RRL equation has a positive water depth bias of 0.03 in. (0.75 mm) and that the Goodyear data are based on total water depth, that is including the average depth of water in the surface texture. When this data bias and problem of term definition are recognized, there is very little disagreement between the equations presented in NCHRP 1-14 for small values of texture. The fact remains that both the RRL and Goodyear equations rest on a much more limited data base than the TTI equation. Therefore, risking the charges of TTI chauvinism the use of this equation is advocated for the purposes of this document.

The compounding effects of transverse cross slope, superelevation and/or longitudinal gradient may result in excessive water depths at certain locations (12). Indications of the effects of some of these variables on drainage length are given by the following equations and detailed in Table 5 for pavement width of 24 ft. (7.9 m).

$$S_3 = \sqrt{S_1^2 + S_2^2} \qquad (19)$$

where

S_3 = slope of resultant flow path in ft./ft. (%)

S_2 = longitudinal gradient in ft./ft. (%)

S_1 = cross slope in ft./ft. (%)

and the length of the flow path is given by:

Table 5. Effect of cross slope and longitudinal gradient on drainage length (11)

Cross Slope S_1	Longitudinal Gradient S_2	Drainage Length L_f	Increase in Drainage Length
ft./ft.	ft./ft.	ft.	Percent
0.0104 (1/8 in./ft.) (1%)	0.000 (0%)	24	-----
0.0104	0.010 (1%)	33	38
0.0104	0.025 (2.5%)	62	158
0.0104	0.050 (5%)	118	392
0.0104	0.100 (10%)	232	867
0.0313 (3/8 in./ft.) (3%)	0.000 (0%)	24	-----
0.0313	0.010 (1%)	25	4
0.0313	0.025 (2.5%)	31	29
0.0313	0.050 (5%)	45	88
0.0313	0.100 (10%)	80	233

Metric conversion: 1 in. = 25.4 mm
1 ft. = 0.305 m

$$L_f = L\left(\frac{S_3}{S_1}\right) = L\sqrt{1 + \left(\frac{S_2}{S_1}\right)^2} \qquad (20)$$

where

L_f = length of flow path in ft. (m/.305)

L = pavement width in ft. (m/.305)

The compounding effect of steep longitudinal gradients and flat cross slopes is clearly evident from the tabulated values. Most road surface types will drain surface water rapidly and reasonably completely if sufficient cross slope is provided but care must be taken to avoid those cross slopes that are not acceptable from general considerations of road safety, geometric design and driveability.

Cross slopes recommended by the American Association of State Highway Officials "Blue Book" (13) are given in Table 6. It may be inferred from this table that cross slopes not less than 1/8 in./ft. (1%) or more than 1/2 in./ft. (4%) are recommended.

Table 6. Recommended AASHO guidelines for normal rural highway pavement cross slopes. (13)

Surface Type	Range in rate of cross slope			
	in./ft.	Ratio	ft./ft.	Percent
High	1/8 - 1/4	1:96 - 1:48	0.010 - 0.020	1.0 - 2.0
Intermediate	3/16 - 3/8	1:64 - 1:32	0.015 - 0.030	1.5 - 3.0
Low	1/4 - 1/2	1:48 - 1:24	0.020 - 0.040	2.0 - 4.0

Metric conversion: 1 in. = 25.4 mm.
1 ft. = 0.305 m.

CHAPTER IV

SURFACE TEXTURE, SKID RESISTANCE AND ACCIDENT FREQUENCY

Although the variables associated with wet pavement accidents are numerous and the interplay among these variables complex, definite progress has been made toward the goal of improved friction. Probably one of the most revealing studies of the effect of pavement friction and wet weather accidents is that of Rizenbergs, Burchett and Napier (14). It is evident from a plot of the data shown in Figure 33 that the accident rate is higher for the lower volume traffic and further there is a definite increase in accident rates for both ranges of traffic volumes studied beginning with skid numbers (SN_{40}) below about 40. If one examines and compares the accident rates for the higher traffic volume on pavements with an SN_{40} of 45 to 50 with that of the lower traffic on roads with an SN_{40} of 25 to 30 one finds that the accident rate has increased more than three times, a jump from about 20 to about 70 accidents per 100 million miles (161 million km) of travel. Since lower traffic volumes usually result in higher average speeds, it is not unreasonable to suggest that the higher death rate on the lower traffic volumed roads would be speed associated. A related study by Holmes, et al. (15), may be used to explain and confirm the findings of Rizenbergs, et al. In Figure 34 it will be noted that both speed and wet pavement conditions contribute to large increases in the accidents due to skidding. Further work by Sabey (16) wherein she compared the percentage of wet road accidents with road friction is shown in Figure 35. As shown in the data of research previously cited, the data of Sabey give a large increase in accidents with a decrease in friction. Sabey has used the British Pendulum Tester to evaluate road friction and in this connection it is interesting to compare the British Pendulum Number (BPN) with SN_{10} as measured by the Texas Highway Department locked wheel skid trailer where water was used as the wetting agent. The relationship developed by Britton (17) is shown in Figure 36. The equation developed is $BPN_{\omega} = 17.5 + 0.87\ SN_{10}$ with $r = 0.99$. In all instances the friction data presented represents full-scale pavements constructed in the usual manner; some were from selected highways while others are full-scale test pads located at Texas A&M. These pads are described in Table 3, Chapter III.

McCullough and Hankins (18) in their presentation of data on fatal and injury accidents from selected sites of study constructed a line as shown in Figure 37 which they chose to call the line of maximum accidents. What appears to be an important conclusion that may be drawn from these and other data presented by the researchers is that for pavements with SN_{50} values of about 45 to 50, the number of fatal and injury accidents decreases rather sharply. Mahone and Runkle (19) have also reported on the variation of wet pavement accidents, relating accident frequency to road friction. Details are given in Figure 38. Again

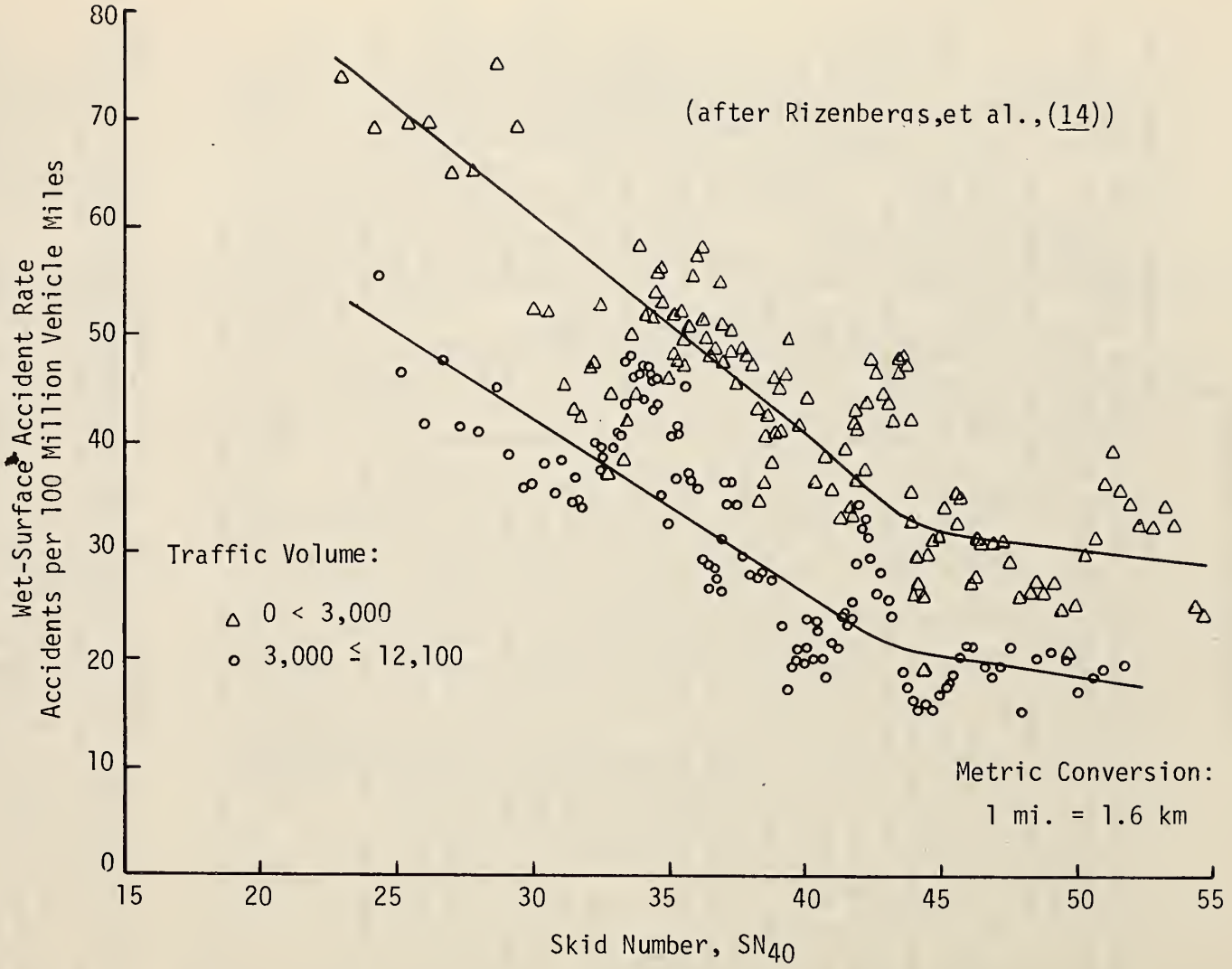


Figure 33. Skid number versus accident rate.

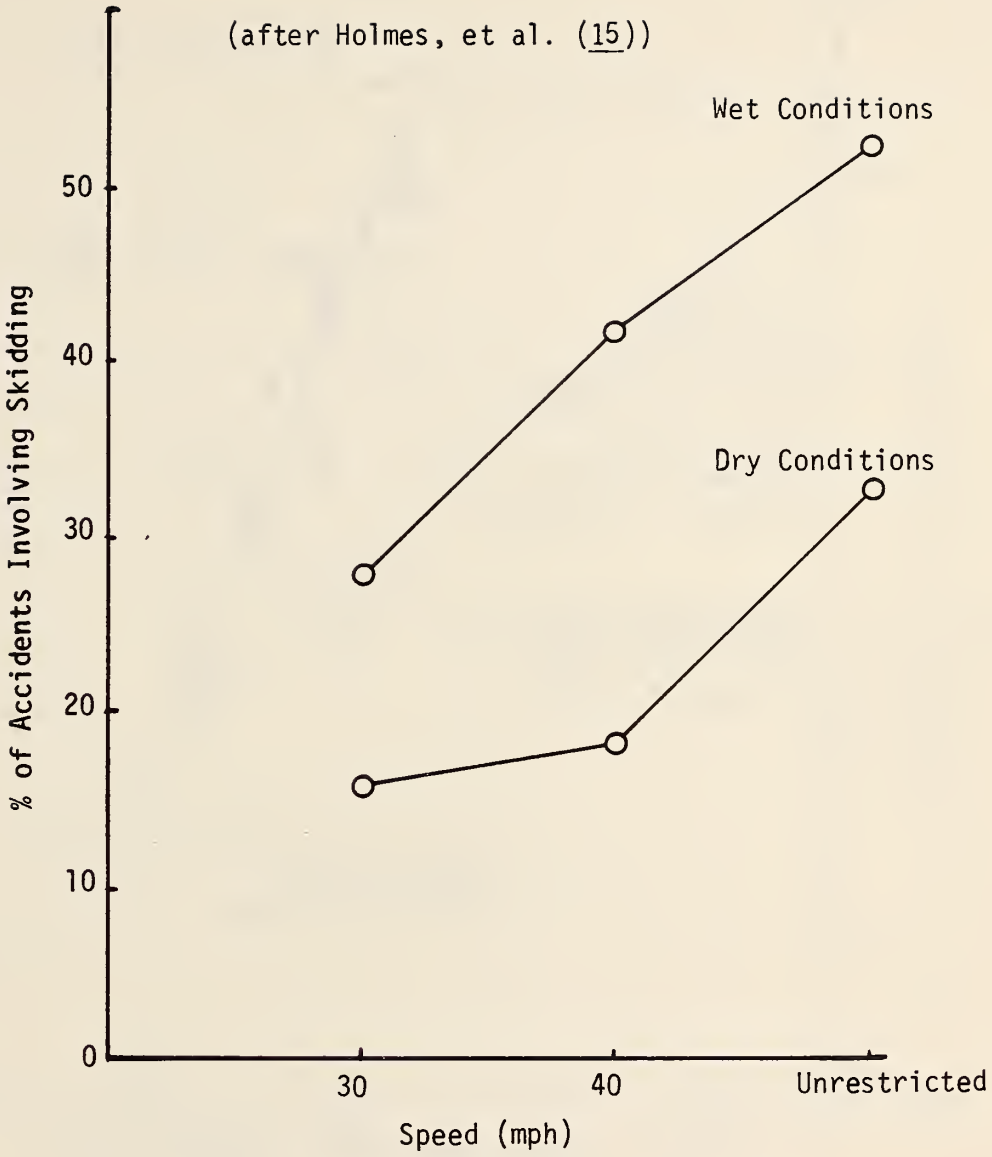


Figure 34. Variation of skidding accidents with vehicle speed and pavement condition.

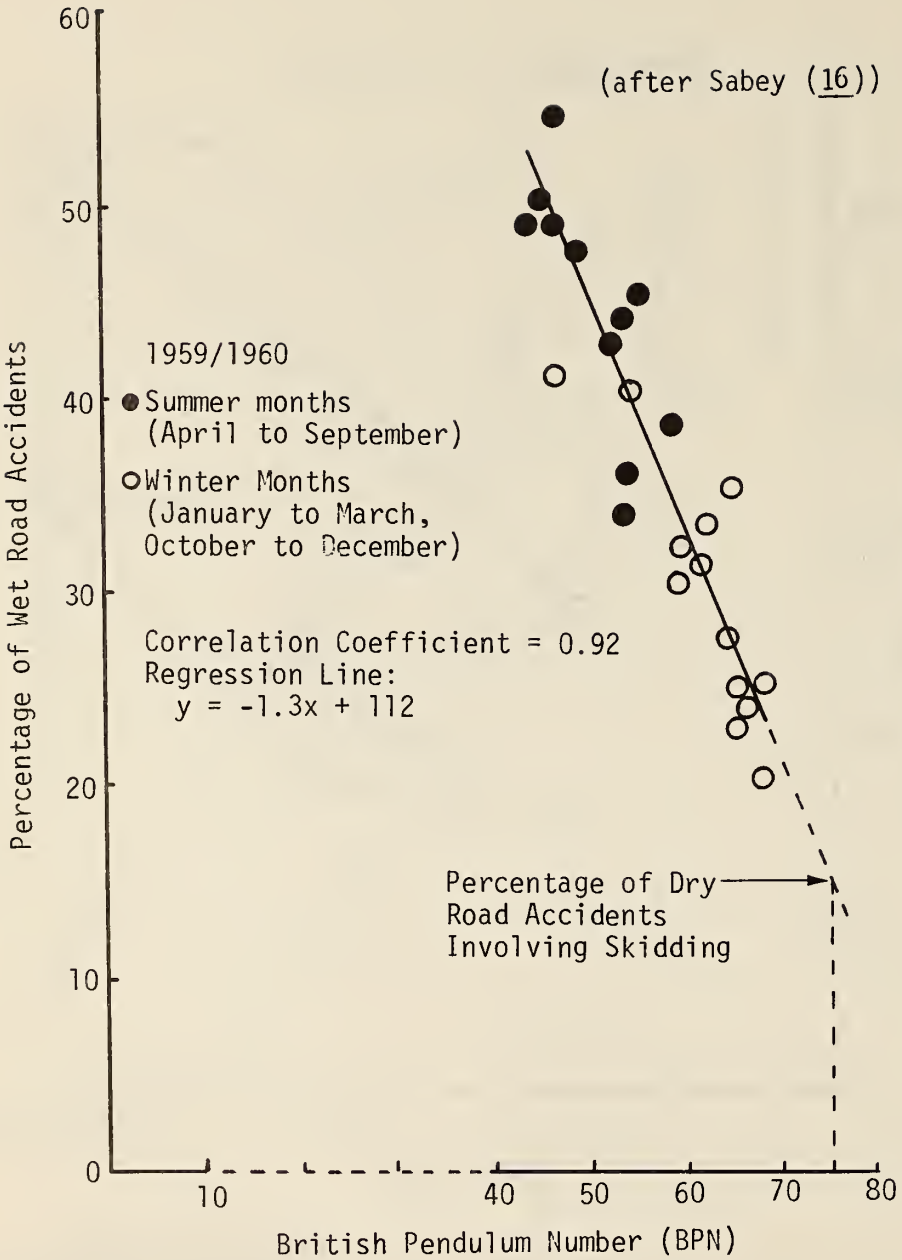


Figure 35. Variation of wet road accidents with measured road surface friction.

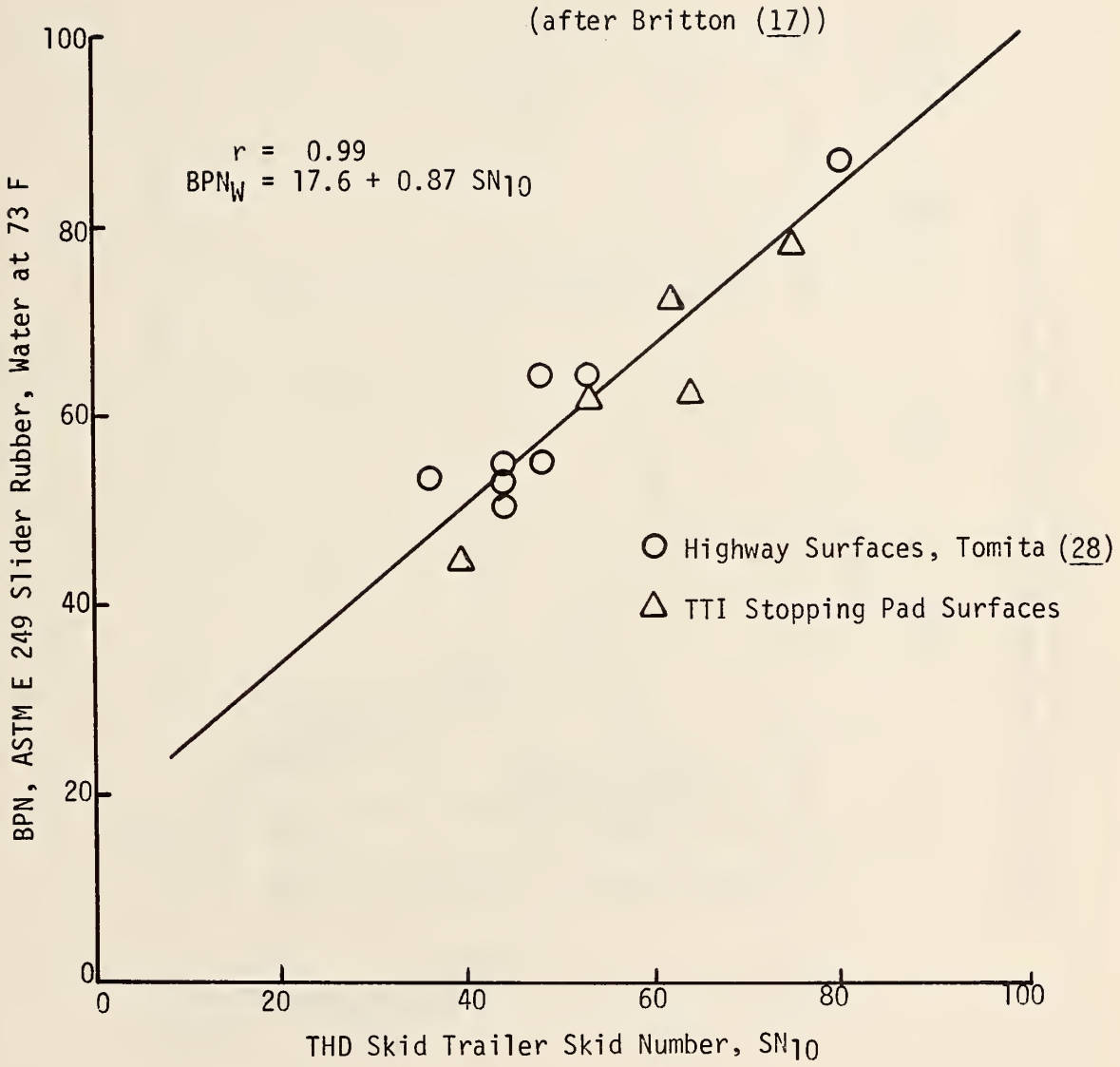
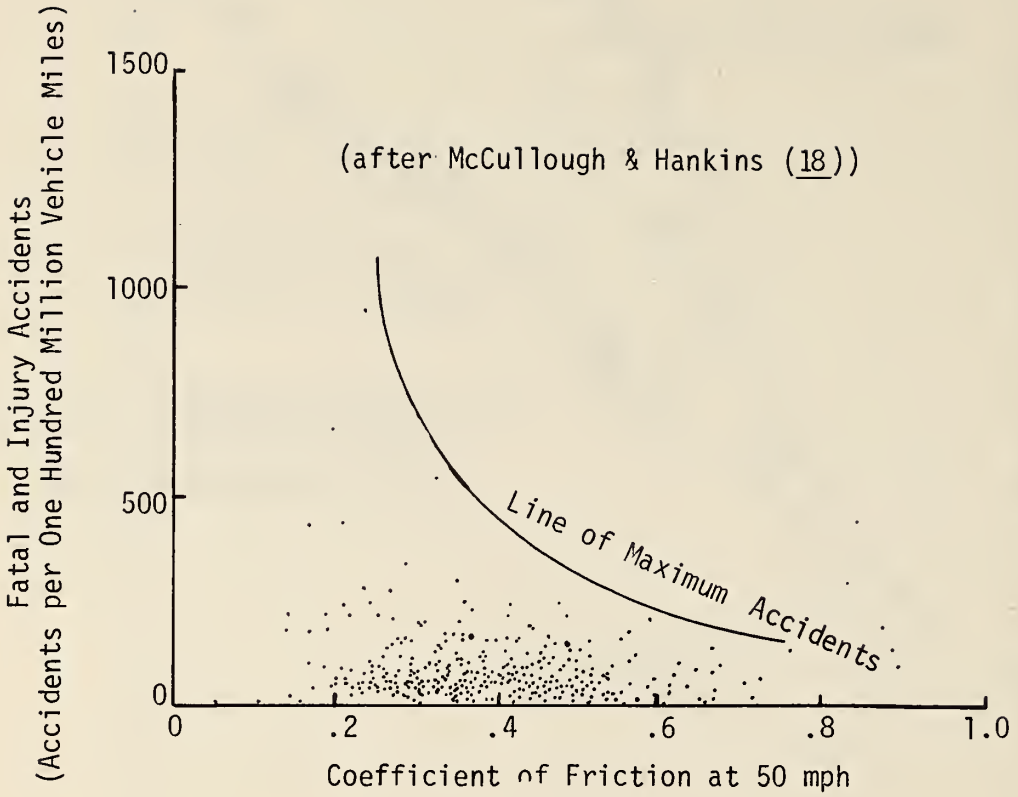


Figure 36. Correlation of BPN (water fluid) with SN_{10}.



Metric Conversion:
1 mph = 1.609 km/h

Figure 37. Comparison of fatal and injury accidents and coefficient of friction at 50 mph.

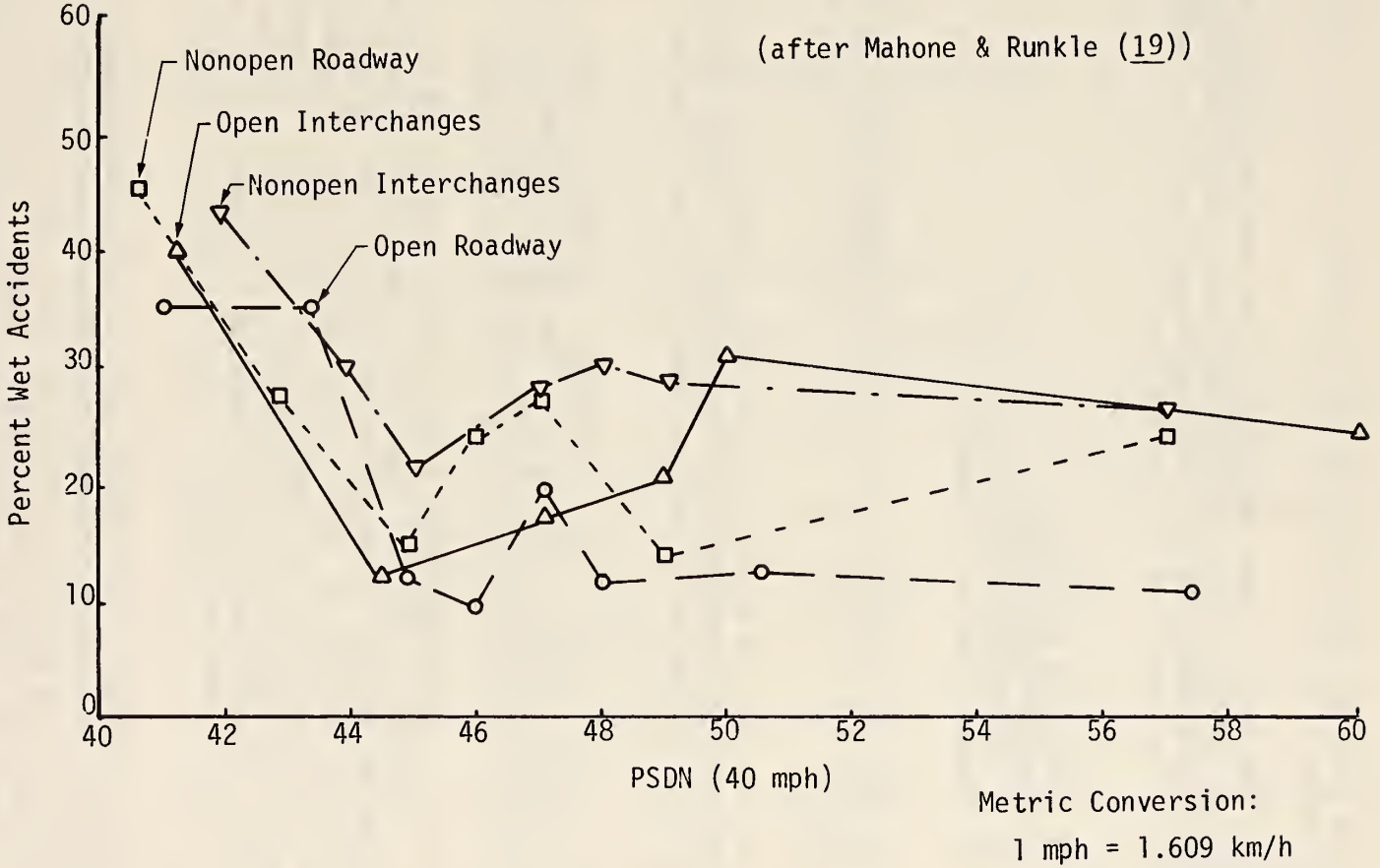


Figure 38. Variation of wet skidding accidents in traffic and passing lanes with stopping distance skid number (PSDN).

it is evident that the break in these curves occurs in the friction coefficient range of 0.40 to 0.50.

Although the total highway accident system involves the driver, the vehicle and the road, only those factors which relate to the road and the interplay between the tire and the pavement will be considered in any detail. The pertinent factors include:

a) Operating conditions
b) Tire characteristics
c) Tire/surface interaction
d) Pavement surface characteristics

Britton (17) has presented a flow chart of these factors and has detailed the interaction among the factors. Figure 39 lists the various relationships.

Factors which may be expected to minimize hydroplaning cut completely across this chart with factors such as water depth, tire properties, drainage and pavement surface characteristics inputting strongly to the situation.

Summarizing from the research previously reviewed it appears evident that for road friction values at or above the SN_{40} range of 40 to 50 the wet pavement accident rate reaches a plateau with only minor decreases in accident rate for friction values above this range.

It is a well established fact that pavement friction or skid resistance is related to the frictional force developed at the tire-pavement interface during acceleration, deceleration and cornering maneuvers (20, 21, 22, 23, 24, 25). Hayes and Ivey (20) from their field experiments evaluated the capability of conventional automobile tire systems to perform a variety of emergency maneuvers. The measurements were compared directly with available friction. Skid numbers and British Pendulum Numbers appear to correlate well with vehicle performance.

Veith and Pottinger (24) reported on a series of tests wherein they measured the relationship between the cornering wet traction coefficient, μ_C, (at any speed) and the water depth with tires of the same tread pattern and two different groove depths. The relationship is shown in Figure 40. The researchers point out--"the linear dependence of μ_C on the log (WD) and the essentially numerically constant differential between tires with full and half tread depth at any selected speed." The authors also point out the possible critical

(after Britton (17))

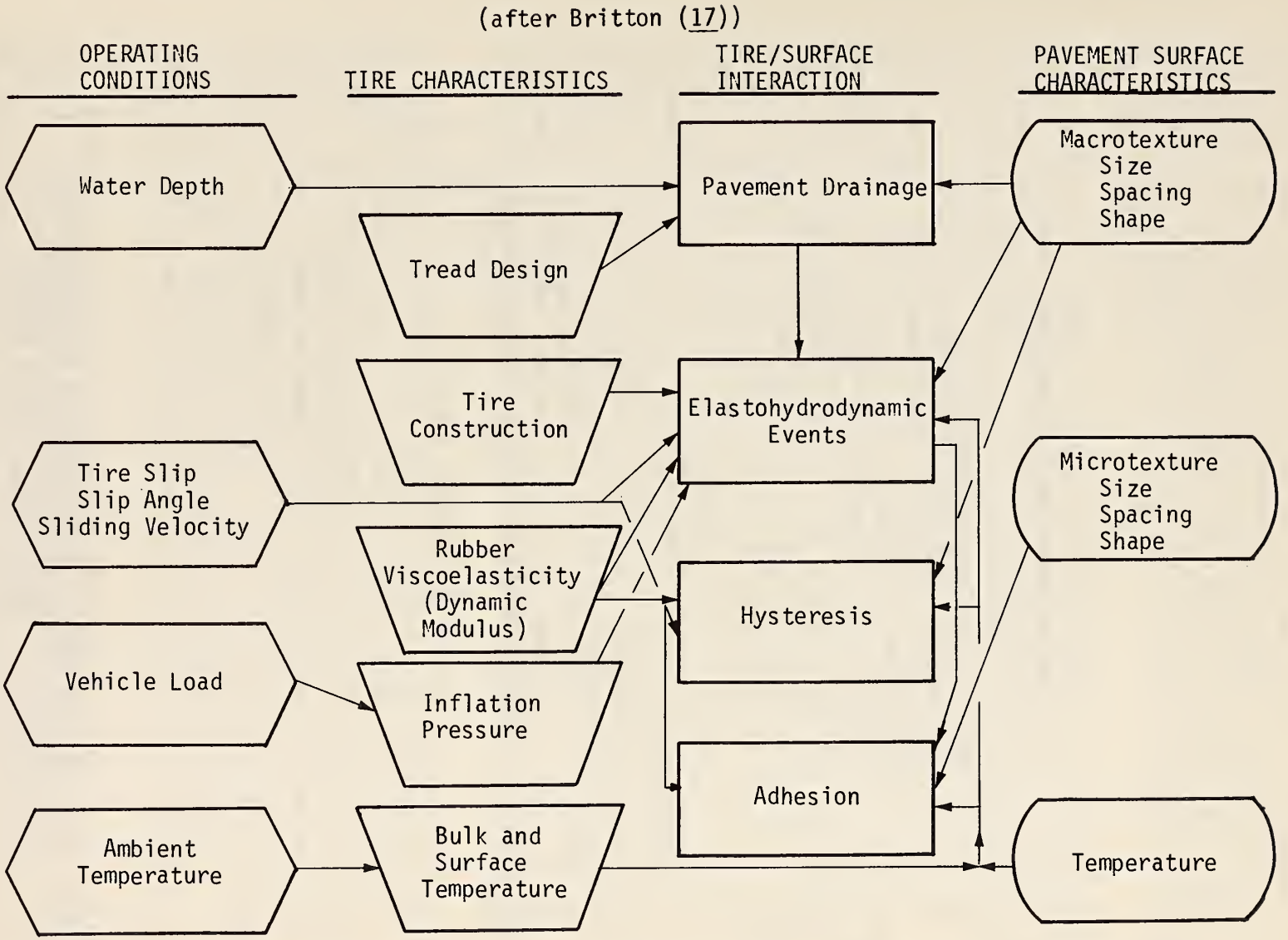
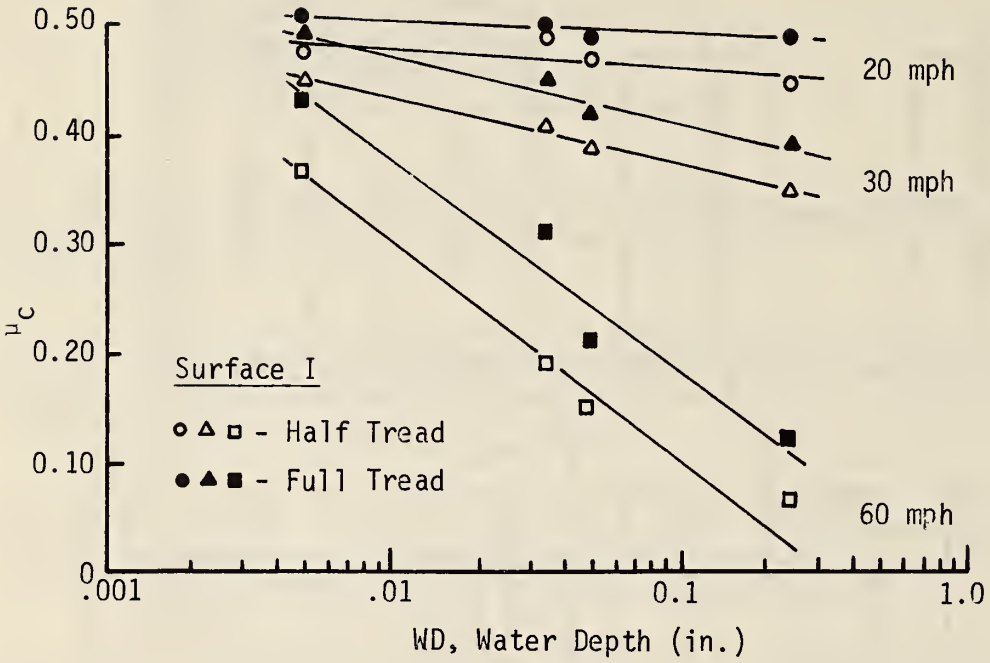


Figure 39. Interrelation among variables influencing automobile skidding on wet pavements.

(after Veith & Pottinger (24))



Metric Conversion:

1 in. = 25.4 mm
1 mph = 1.609 km/h

Figure 40. The influence of water depth on tire cornering wet traction coefficient for tires of the same tread pattern in two different groove depths.

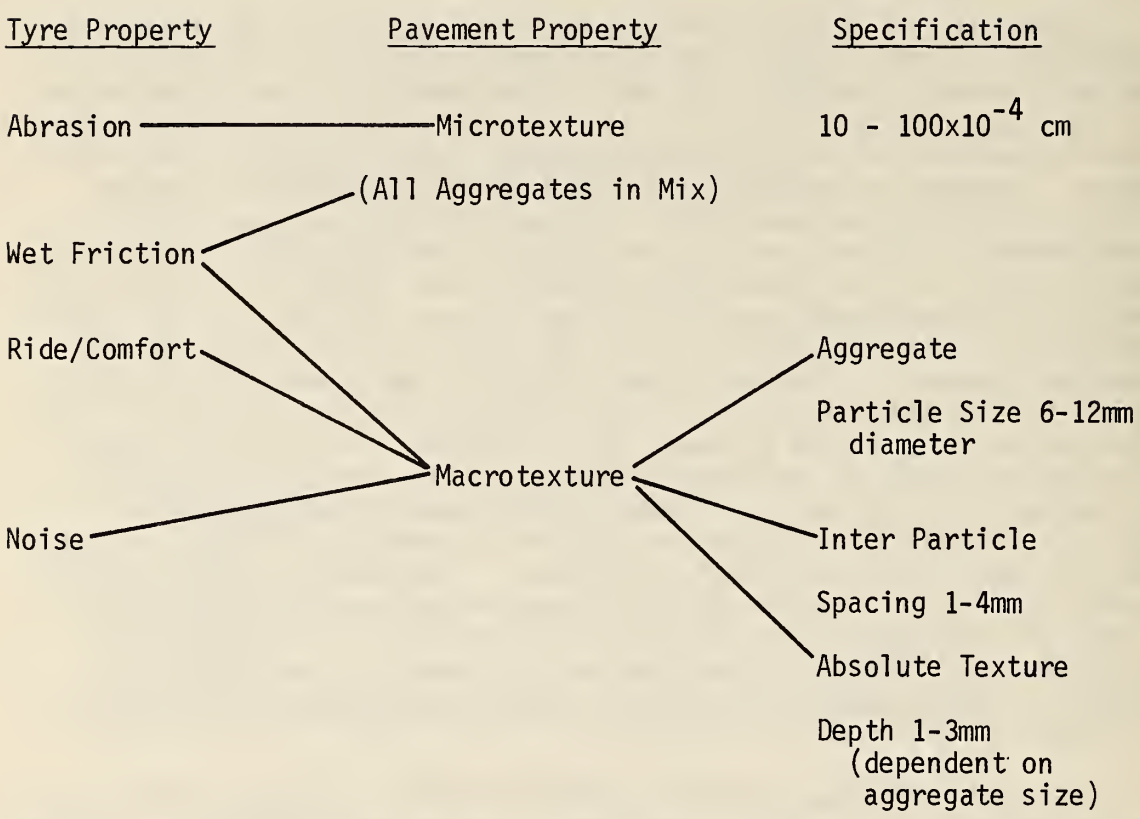
effect of wind velocity and direction on water depths. Differences in the order of a factor of three or more are quoted. Reference to Figure 40 makes it quickly evident that wind direction and velocity may constitute a critical input to the tire-pavement interaction. This would suggest a pavement texture and cross-slope combination that would minimize the probability of positive water depths on pavements subject to high-speed traffic.

Bond, et. al. (25), in their concluding remarks on the effective design of pavement texture and tire-pavement interaction presented the criteria given in Table 7. In their experiments with the optimization of the road surface characteristics they stress the critical importance of microtexture for the development of wet friction at all levels of speed. Further consideration was given to the relationship between microtexture size and tire wear. Separately from and essentially concurrently with the work of Bond, et. al., Britton (17) investigated microtexture size as this factor relates to wet pavement friction. The results of both studies are surprisingly close. Britton did not study the effect of microtexture on tire wear but instead used the British Pendulum Number to study the effect of adding microtexture of various sizes to 4 mm glass spheres. The results are shown in Figure 41. Returning now to the work of Bond, et. al.,one may look at the relationship of tire wear and microtexture size. Their summarized data are presented in Figure 42. Comparing the peak (about 10 to 100 microns) of Britton's (17) curve with the selected band shown in Figure 42, an excellent correlation is evident. Tire wear is a very definite consideration that should not be disregarded when setting the upper limit of microtexture. At the same time, however, if a reduction in the number and severity of wet weather accidents can be effected by enhancing this factor, microtexture scaled above these "optimums" may very well be in order.

However, if enough water is present to produce dynamic hydroplaning, microtexture will be of no significant value in the development of tire-pavement tractive forces. Pavement surface drainage is an absolute additional requirement for wet weather use by high speed traffic and is usually considered to be supplied by macrotexture in combination with adequate cross slope; although water escape at the tire-pavement interface may be brought about by drainage into the surface layer of the pavement via interconnected voids. Both of these water escape mechanisms may be aided by water escape grooves in the tread pattern of the tire. Different combinations of these water escape devices may function simultaneously to maximize the available friction on a given facility.

Reference is made to the research reported by Bond, et. al. (25), wherein a macrotexture size range of 6 to 12 mm was specified. Balmer (26) in a paper dealing with the significance of pavement texture gave an upper value of 15 mm. Some state highway agencies are placing surface course mixtures which contain aggregates measuring 20 mm. The studies conducted by Rose, Gallaway and Hankins (27) included typical

Table 7. Pavement textural characteristics required for optimum tyre performance.



(after Bond, et al. (25))

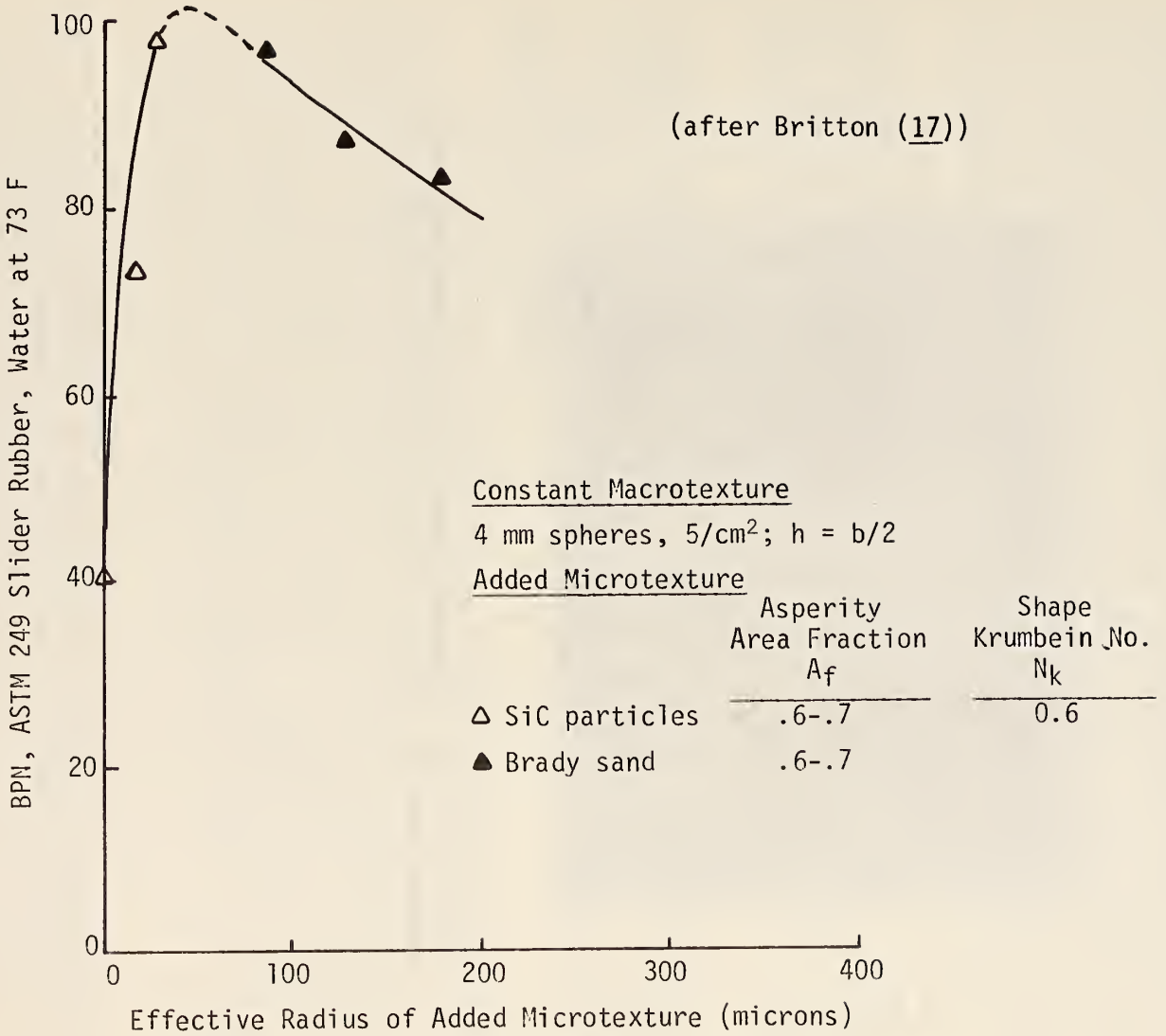


Figure 41. Variation of British Pendulum Number (BPN) with microtexture size and shape added to 4 mm glass spheres.

(after Bond, et al. (23))

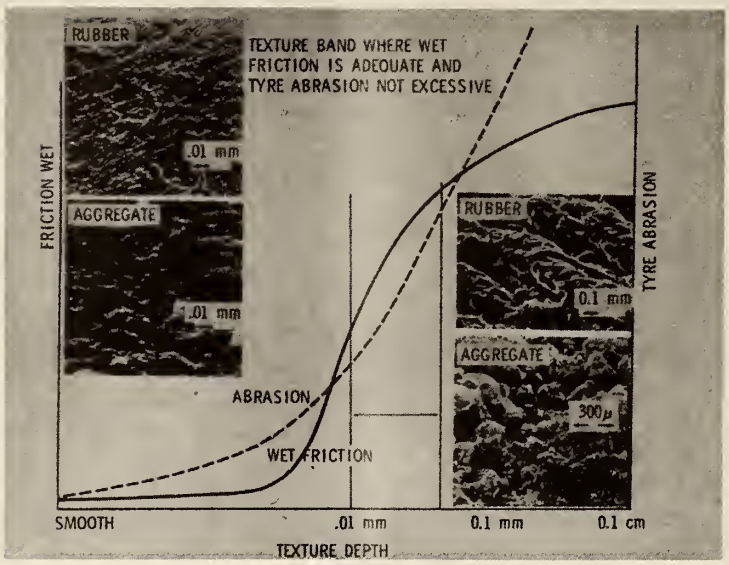


Figure 42. The increase in wet friction and tyre tread compound abrasion with increase in aggregate micro-texture.

surfaces used by the Texas Highway Department. The vast majority of the aggregates in these surfaces were 15 mm or less in top size. The general consensus is that for pavement macrotexture much above a top size of 15 to 20 mm road noise becomes a factor of concern (26).

A survey by Tomita (28) covered nineteen different methods used to measure pavement surface texture. Rose, Hutchinson and Gallaway (29) compared these and other methods of texture measurement for reliability, repeatability, cost, operational ease and correlation with skid resistance measurements. Of all the methods compared only the Schonfield stereophoto-interpretation method (30) measures texture in terms of basic parameters and combines these parameters in an overall scheme to predict skid numbers. This system is currently being improved to overcome some limitations that have been found in the procedure.

One other method reviewed in reference (29) that has merit for evaluating asperity geometry is one developed by Moore (31). A drainage meter was used to determine the mean hydraulic radius of the flow path between the tire and the pavement. Moore indicated that it was possible to relate this value with water removal from under the tire when one also considered measurements of macrotexture size and shape.

In a current study with the Texas Highway Department and the Federal Highway Administration TTI researchers have made limited and indirect measurements of the wet weather performance of open graded surfacings with a water outflow device (32). From the data collected it appears that outflow measurements are more reliable for predicting tire-pavement interface drainage than are macrotexture measurements made with silicone putty.

Still another aspect of texture is evident when one examines the data from two somewhat similar surfaces tested under simulated rainfall conditions (33). The two surfaces in question are both chip seals, Pad No. 6 a round gravel and Pad No. 7 a partially crushed synthetic aggregate. The rounded gravel at the time of testing had a measured texture of 0.159 inches (4.0 mm); whereas, the synthetic aggregate surface measured 0.136 inches in (3.5 mm). Skid tests under simulated rain at 20, 40 and 60 mph (32, 64, 97 km/h) indicated much higher SN values for Pad No. 7, the pad with less measured texture. From a simple analysis of the parameter values one might predict other than the SN values obtained.

Consider the water depths recorded during the tests on Pads Nos. 6 and 7. Comparison of these figures indicates a marked difference in the drainage characteristics of the two surfaces. The range of rainfall intensities for the two surfaces was about the same; however, in the case of Pad No. 6 water depths ranged from -2.21 to +0.77 mm, whereas the water depth on Pad No. 7 ranged from -0.50 to +1.60 mm. If one averages these depths, the result is -1.44 mm for Pad No. 6 and +1.10 mm for Pad No. 7. Yet, Pad No. 7 gave much higher SN values.

Particle shape and surface texture are suggested as the key factors explaining these observed differences. The gradings of the two aggregates used in these seal coats were essentially the same when they were constructed, but because the crushed synthetic or manufactured aggregate of Pad No. 7 degraded somewhat due to construction rolling and subsequent service, and since there was only limited degradation of the natural rounded siliceous gravel, Pad No. 6 finally achieved a higher texture value. The drainage of Pad No. 6 was better than that of No. 7 for these reasons, namely, larger and smoother drainage channels. The higher SN values for Pad No. 7 may very well have merit, but one may also wish to consider another performance characteristic of this type of surface; this being splatter and spray. Pad No. 6 would be capable of draining large amounts of water for normal cross slopes of 1/8 to 1/4 in./ft. (1% to 2%) without appreciable splatter or spray. This is particularly important for night driving on two-lane highways and should not be completely disregarded on multilane divided highways.

How then does one weigh the advantage of higher SN values? Strictly speaking, other factors must be taken into account, because it is the combined effect on the total accident picture that must be considered and the interactions are numerous and often rather difficult to separate for analysis; yet, there must be an answer. Indeed, there must be not one but several answers, each tailored for a particular but somewhat general set of conditions and service demands. The number of categories of pavement surface textural types desirable for a given area or region will probably vary, but most likely three or four will suffice.

It is evident from the data presented that levels of performance, as defined by SN values, may be produced via several different types of surfaces. This being the case, it would seem that ease of construction, durability, uniformity and long term cost would become determining factors in the selection of the surface to be built.

Since driver expectancy (73, 74) is a critical input to overall highway safety, one should examine the effect of the SN value on this factor and consider this in the selection of the surface to be built. Probably the most important input to the driver expectancy-accident related phenomena is uniformity, that is, uniformity of tire-pavement interaction feedback to the driver. Knowledge of this fact is a strong input when the decision is made concerning how many general categories of surfaces will be made available to the driving public and what will be the range of SN value furnished in each category.

Ideally, the variation in tire-pavement interaction feedback to the driver should be as small as practical. Any sudden change in the SN value, although it may be relatively small, could conceivably be a causative factor in an accident. The driver might be unable to adjust to the new hazard in the very short time available for an appropriate reaction, if he has only recently entered a section of road with a significantly different SN value. This could be true whether the newly entered section has a higher or lower SN value than the surface

to which he has become acclimated. Admittedly, change from a higher SN to a lower SN offers greater probability for an accident.

This concept would most likely discourage the use of pavements with very high SN values in favor of a system of streets and highways with intermediate ranges of SN values and pavements with relatively flat SN speed curves, and encourage the early elimination of surfaces with low SN values and/or steep SN speed curves. Of course, speed zoning could eliminate the need to avoid the latter, but quite often a pavement that is very speed sensitive is also sensitive to the wet condition, and therefore such a pavement would not be desirable even if the facility were speed zoned.

A comprehensive body of data has recently been developed which relates the skid number at different speeds to surface type and texture and to the degree of wetness, as indicated by average water depths. This information is separated into two categories, the first relating primarily to asphaltic pavements (random textures) and the second to portland cement concrete pavements (patterned textures) (33).

Random Textured Pavements

Skid trailer tests under the rainfall simulator shown in Figure 3, page 6, were performed on seven different asphalt and two concrete test pads at Texas A&M during 1971 and 1972 (33). These tests were recently reanalyzed using linear regression techniques.

The variables that were measured were:

1. The skid number, SN.
2. The pavement macrotexture as measured by the silicone putty test, TXD in in. (mm/25.4).
3. The tire tread depth, TD, in 1/32 in. (.79 mm).
4. The water depth, WD, in mm and converted to in. for use in the equations.
5. The test speed, V, in mph ((km/h)/1.609).
6. The tire pressure, 24 psi and 32 psi (kPa/6.895).
7. Texture direction on concrete pavements, transverse and longitudinal.
8. The air temperature, some water film temperatures, some wind velocities and directions were measured but none of these variables were used in the regression analyses.
9. No measurements were made of the pavement microtexture.

The results of these tests are summarized in Figures 43 through 47. In all cases the skid number decreases with increasing speed showing the progressive influence of partial hydroplaning.

Figure 43 shows the effects of pavement macrotexture, tire tread depth, water depth, and speed on the skid number. The highest skid number is produced by the combination of maximum texture depth and

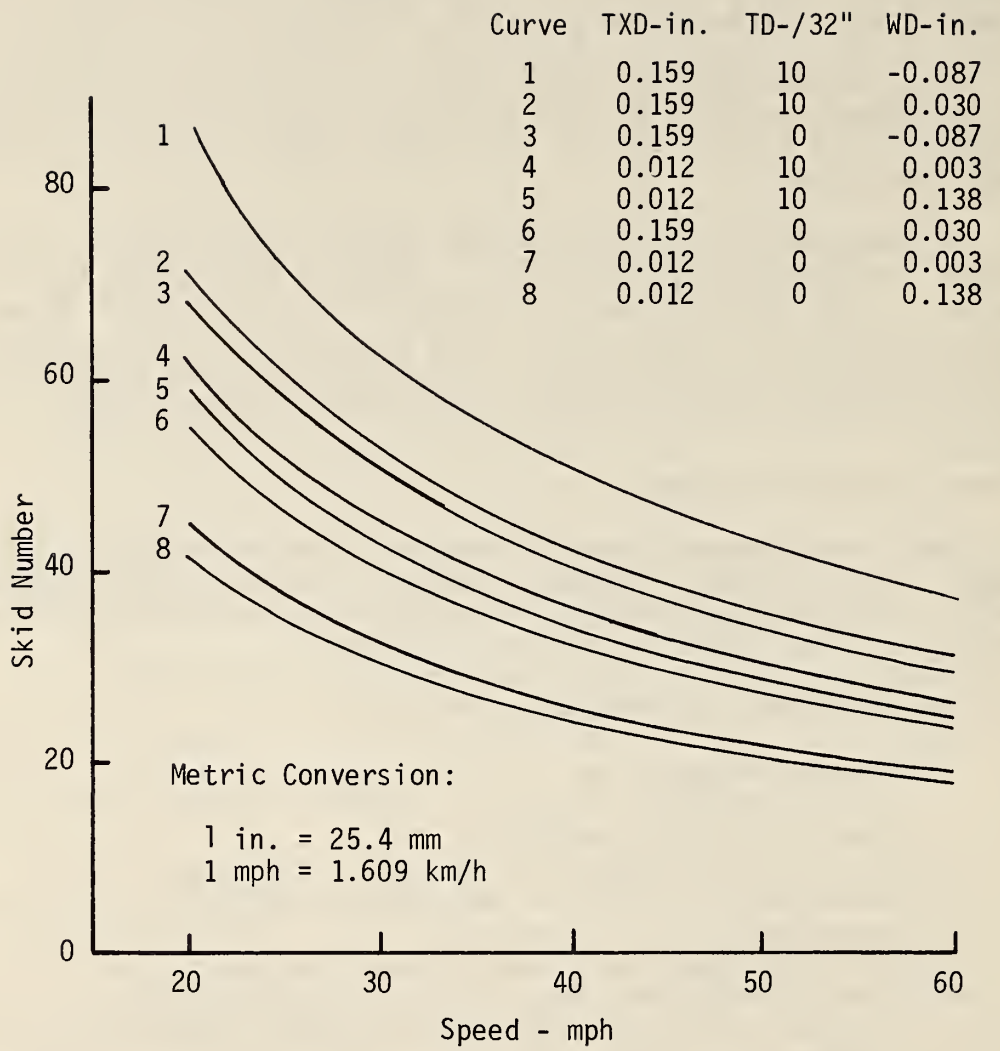


Figure 43. Effects of pavement texture depth (TXD), tire tread depth (TD), and water depth (WD) on the skid number (SN) at speeds of 20 mph to 60 mph (32 to 97 km/h), ASTM-14" (356 mm), 24 psi (165.5 kPa) tire inflation pressure.

tread depth and minimum water depth. This combination of texture and tire tread depth allows for maximum fluid escape paths through the tire grooves and pavement surface voids between tire and pavement. At the other extreme, a minimum macrotexture pavement, a minimum tire tread depth, and a maximum water depth give the lowest skid numbers. The flow paths are much restricted resulting in a more pronounced hydroplaning effect throughout the speed range.

The effect of tire inflation pressure is illustrated by Figures 44 through 47. Examination of Figures 44 and 45 will show that for low macrotexture pavements, increased tire pressure results in increased skid numbers at minimum and maximum water depths. If the macrotexture is large, tire inflation pressure has a negligible effect on the skid number, both for minimum and maximum water depths. See Figures 46 and 47.

Although a single equation to predict skid number as a function of all influencial parameters would be preferred, efforts to develop such an equation resulted in a relatively low correlation with the empirical data. Thus it was found necessary to use a multiple equation approach with different exponents and constants appropriate for different tire and surface characteristics. Admittedly, the utility of the equations is limited because of this lack of generality.

A summary of the equations developed to predict skid numbers follows:

Equations Used for Annex Data (Pads 1, 3, 4, 5, 6, 7, 8)

(TXD varies from 0.002 to 0.036 in. (0.051 to 0.914 mm))

where

SN = Skid number
V = Speed in mph ((km/h)/1.609)
TD = Tire tread depth in 1/32 in. (mm/25.4)
TXD = Texture depth, silicone putty test, in in. (mm/25.4)
WD = Depth of water film above tops of pavement asperities in in. (mm/25.4)

This value may be negative.

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{154}{V^{0.77}} \left[TD^{0.05} + \frac{4.71\ TXD^{0.09}}{(25.4\ WD + 2.5)^{0.09}} \right] \quad (21)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{135}{V^{0.72}} \left[TD^{0.06} + \frac{4.18\ TXD^{0.05}}{(25.4\ WD + 2.5)^{0.08}} \right] \quad (22)$$

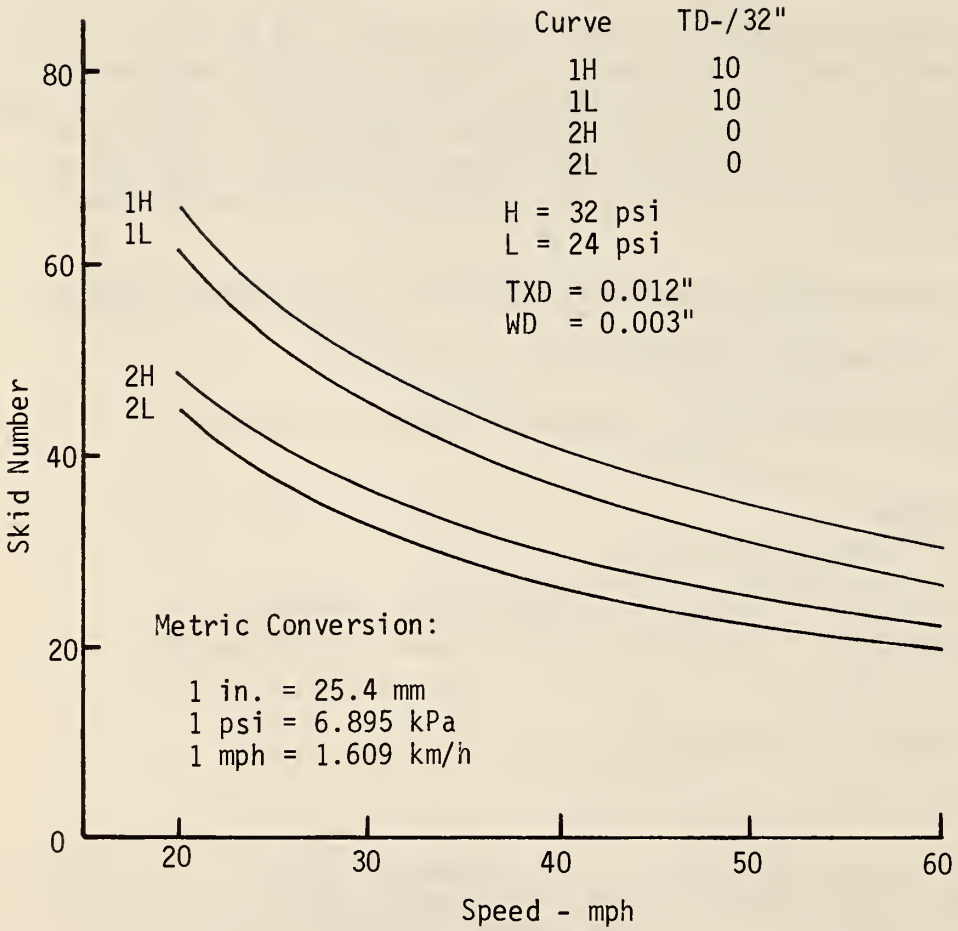


Figure 44. Effect of tire inflation pressure on skid numbers (SN) for the ASTM 14" (356 mm) tire at minimum water depth (WD), minimum and maximum tire tread depth (TD), on low texture depth pavement (TXD).

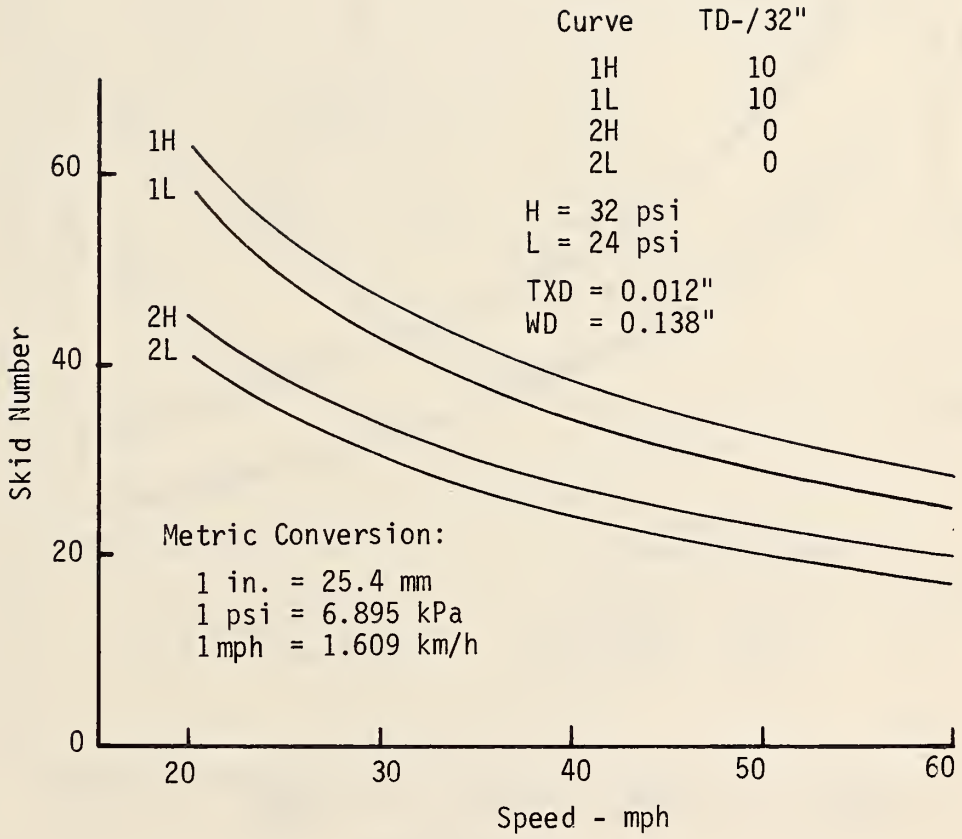


Figure 45. Effect of tire inflation pressure on skid numbers (SN) for the ASTM 14" (356 mm) tire at maximum water depth (WD), minimum and maximum tire tread depth (TD), on low texture depth pavement (TXD).

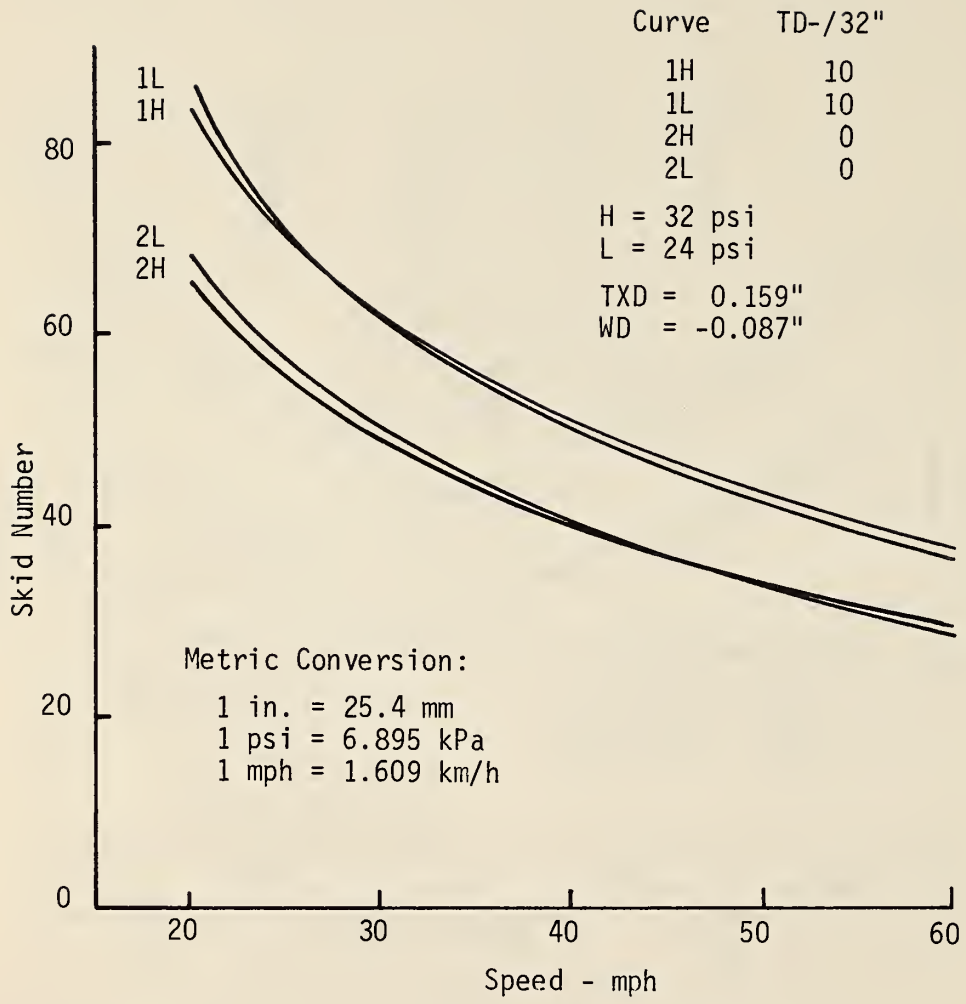


Figure 46. Effect of tire inflation pressure on skid numbers (SN) for the ASTM 14" tire at minimum depth (WD), minimum and maximum tire tread depth (TD) on high texture depth pavement (TXD).

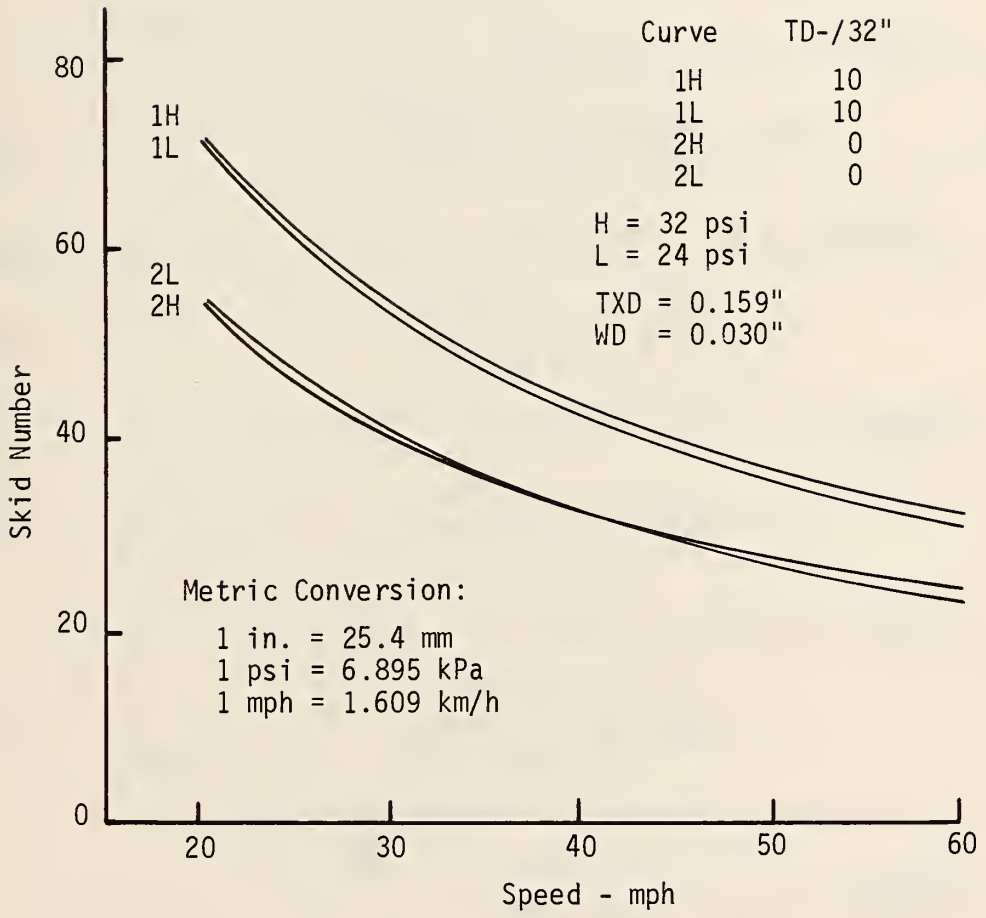


Figure 47. Effect of tire inflation pressure on skid numbers (SN) for the ASTM 14" (356 mm) tire at maximum water depth (WD), minimum and maximum tire tread depth (TD) on high texture depth pavement (TXD).

Commercial 24 psi (165.5 kPa)

$$SN = \frac{234}{V^{0.86}} \left[\frac{TD^{0.06}\ TXD^{0.07}}{(25.4\ WD + 2.5)^{0.13}} + 2.08 \right] \quad (23)$$

Commercial 32 psi (220.6 kPa)

$$SN = \frac{186}{V^{0.83}} \left[\frac{TD^{0.07}\ TXD^{0.01}}{(25.4\ WD + 2.5)^{0.16}} + 2.49 \right] \quad (24)$$

Equations for Pad 2 Jennite Flush Seal TXD = 0.002 in. (0.051 mm)

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{261\ (TD + 1)^{0.125}}{V^{0.851}\ (25.4\ WD)^{0.072}} \quad (25)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{248\ (TD + 1)^{0.157}}{V^{0.824}\ (25.4\ WD)^{0.028}} \quad (26)$$

Commercial 24 psi (165.5 kPa)

$$SN = \frac{316\ (TD + 1)^{0.189}}{V^{0.93}\ (25.4\ WD)^{0.24}} \quad (27)$$

Commercial 32 psi (220.6 kPa)

$$SN = \frac{281\ (TD + 1)^{0.166}}{V^{0.873}\ (25.4\ WD)^{0.206}} \quad (28)$$

Equations for Pad 9 Painted Concrete TXD = 0.033 in. (0.838 mm)

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{583\ (TD + 1)^{0.241}}{V^{0.988}\ (25.4\ WD)^{0.015}} \quad (29)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{506\ (TD + 1)^{0.219}}{V^{0.928}\ (25.4\ WD)^{0.116}} \qquad (30)$$

Commercial 24 psi (165.5 kPa)

$$SN = \frac{811\ (TD + 1)^{0.312}}{V^{1.152}\ (25.4\ WD)^{0.031}} \qquad (31)$$

Commercial 32 psi (220.6 kPa)

$$SN = \frac{657\ (TD + 1)^{0.301}}{V^{1.069}\ (25.4\ WD)^{0.037}} \qquad (32)$$

Equations for Pad 1 Portland Cement Concrete TXD = 0.036 in. (0.914 mm)

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{62}{V^{0.85}} \left[8.22\ (TD + 1)^{0.12} + \frac{1}{(25.4\ WD)^{0.07}} \right] \qquad (33)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{126}{V^{0.81}} \left[3.15\ (TD + 1)^{0.12} + \frac{1}{(25.4\ WD)^{0.05}} \right] \qquad (34)$$

Commercial 24 psi (165.5 kPa)

$$SN = \frac{179}{V^{0.91}} \left[2.28\ (TD + 1)^{0.19} + \frac{1}{(25.4\ WD)^{0.07}} \right] \qquad (35)$$

Commercial 32 psi (220.6 kPa)

$$SN = \frac{258}{V^{0.92}} \left[1.48\ (TD + 1)^{0.21} + \frac{1}{(25.4\ WD)^{0.08}} \right] \qquad (36)$$

Patterned Texture Pavements

The surface macrotexture of newly constructed PCC pavements is generally patterned, rather than random, and the patterns - or striations - are generally imparted to the concrete while it is still fresh by dragging such devices as burlap cloth, brooms, or metal tines along the surface (34). The striations can be placed either longitudinally (parallel to the direction of travel) or transversely (perpendicular to the direction of travel).

On worn PCC pavement surfaces, any new macrotexture is usually imparted by grooving the pavement surface (35). These grooved textures are also patterned - rather than random - and may be either transverse or longitudinal. Therefore, any investigation of the texture requirements of PCC pavements must consider the effects of a patterned texture and the direction of these patterns (longitudinal or transverse).

Although the importance of proper texturing has been recognized for a number of years (36), the literature on PCC texturing is not extensive. Two summaries of the state of the art on PCC pavement textures (34, 37) deal almost exclusively with pavements textured with a longitudinal burlap drag. The basic literature establishing the methodology and importance of skid resistance also utilized data gathered on longitudinal burlap drag surfaces (6, 38).

Only recently have data been gathered on skid resistance of PCC pavement textures other than longitudinal burlap drag. Noteworthy is the work done in England (39) where it was concluded that transverse texturing is superior to longitudinal texturing and plastic grooving is the best method of texturing. Work reported in North Carolina (40), Missouri (41), Connecticut (42, 43), and Texas (44) indicate that certain new textures such as the metal tines offer promise as an improved texture and that transverse texturing appears to be superior to longitudinal texturing. A recent survey by HRB Committee A2F01 indicates that several states are now using various methods to achieve deeper textures with correspondingly higher skid number (46). A comprehensive study in Georgia was recently reported by Thorton (45). As a result of this study, Georgia has published a Special Provision which requires a steel tine finish on PCC surfaces to produce from .035 in. (0.89 mm) to .060 in. (1.52 mm) texture.

Most of the referenced reports deal with improvement in skid resistance as measured by the standard skid trailer utilizing its internal watering system (ASTM E274), which does not seem to adequately simulate potential hydroplaning conditions. Thus conclusions, or even inferences, on factors affecting hydroplaning are difficult to draw from the majority of the research on skid resistance. Some very significant work was reported by Horne and Dreher on aircraft tires (3) in which they indicate some procedures - including transverse grooving - for minimizing the danger

of hydroplaning. Other important contributions include the works by Horne and Joyner (4), Sabey, et al. (47), Horne et al. (48), and Stocker and Lewis (49). These reports indicate the seriousness of the problem and those dealing with striation direction (4, 48) indicate transverse striations minimize hydroplaning conditions more than longitudinal texturing. In terms of texture criteria the available information is sketchy, as tests were conducted on pavement surfaces where texture parameters were not varied over a very wide range.

An extensive study of skid resistance under simulated rainfall on patterned PCC pavements was done under HPR Study 2-6-70-141 (50). This study evolved 17 full scale test sections which are described in Table 8. The geometric layout of the sections is shown in Figures 48 and 49.

Skid measurements under simulated rainfall are given in Tables 9 and 10. Note that both "bald tire" data and "treaded tire" data are given. The treaded tire is the standard ASTM tire (ASTM E249) with 11/32 in. (8.7 mm) tread depth, while the bald tire is the same ASTM tire reduced to a tread depth of 2/32 in. (1.6 mm). All of the data in Tables 9 and 10 were statistically analyzed using a two-step select regression analysis technique where best fit models were developed. The first step was to develop models in natural log form:

$$SN = \frac{e^{A_1} (TD + 1)^{A_2} TXD^{A_3}}{V^{A_4} (WD + 0.1)^{A_5}} \quad \ldots\ldots\ldots\ldots (37)$$

where

e = base of the natural logarithm
SN = skid number
V = vehicle speed in mph ((km/h)/1.609)
TD = skid tire tread depth in 32nds in. (0.79 mm)
TXD = pavement texture depth in in. (mm/25.4), based on putty impression values (50)
WD = water depth on the pavement surface in in. (mm/25.4) measured from the top of the pavement asperities
A_1, A_2, A_3, A_4, A_5 = constants

The second step was to utilize the best fitting log models and statistically combine variables to develop models of the following form:

$$SN = \frac{C_1}{V^{C_2}} \left[C_3 (TD + 1)^{C_4} TXD^{C_5} + \frac{1}{(WD + 0.1)^{C_6}} \right] \quad \ldots\ldots (38)$$

Table 8. Surface finishes. (50)

Finish Type	Description	Test Section No.
Broom	Accomplished by passing a plastic-bristle broom over the slab surface, slightly grooving the concrete. The broom was inclined at an angle of approximately 30 degrees to the surface.	F-1, F-3
Burlap Drag	A burlap drag finish is accomplished by passing a wet burlap cloth, with approximately two ft. of burlap in contact with the surface until the desired texture is obtained.	F-6, F-17
Brush	Accomplished by passing a natural-bristle brush (strawlike) over the slab surface, slightly grooving the concrete. The broom is inclined at an angle of approximately 30 degrees to the surface.	F-7, F-20
Tines	Accomplished by passing a series of thin metal strips (tines), 1/8 in. by 5 in. long (3 mm by 130 mm) over the section surface, producing grooves of approximately 1/8 in. (0.3 cm) depth in the concrete. The tine spacing was varied from 1/8 in. to 1/4 in.	F-2, F-4, F-15, F-16, F-18, F-19
Burlap Drag Plus Longitudinal Tines	Accomplished by first passing burlap drag over the section surface, followed by one pass of the tines. 1/8 in. by 5 in. long (3 mm by 130 mm), over the section surface, producing grooves of approximately 1/8 in. (3 mm) depth in the concrete. The tine spacing was varied from 1/4 in. to 1.0 in. (6 mm to 25 mm). (clear distance between tines).	F-5, F-11, F-12, F-13, F-14, F-21

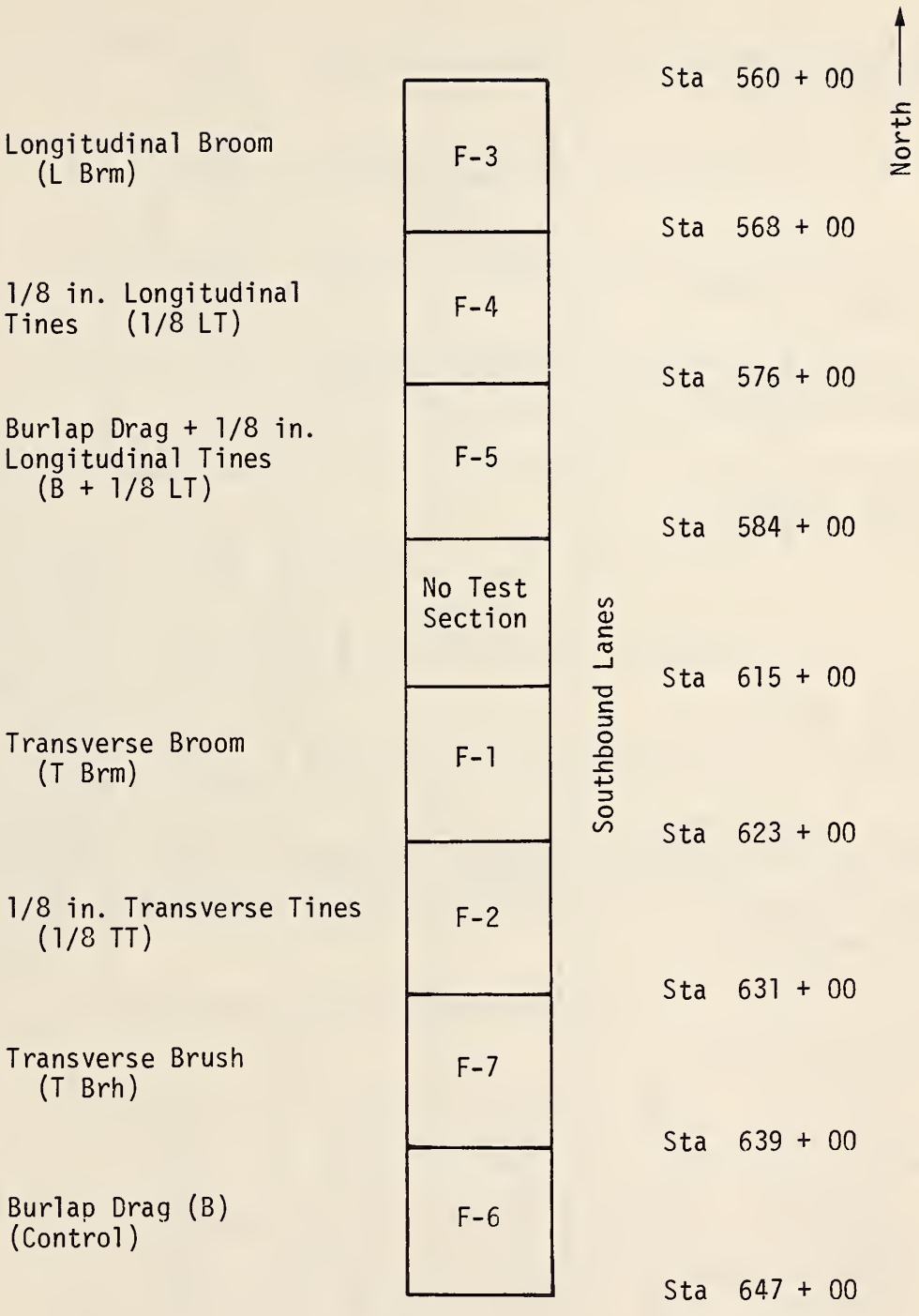


Figure 48. Location of test sections, SH 6 in College Station, Texas

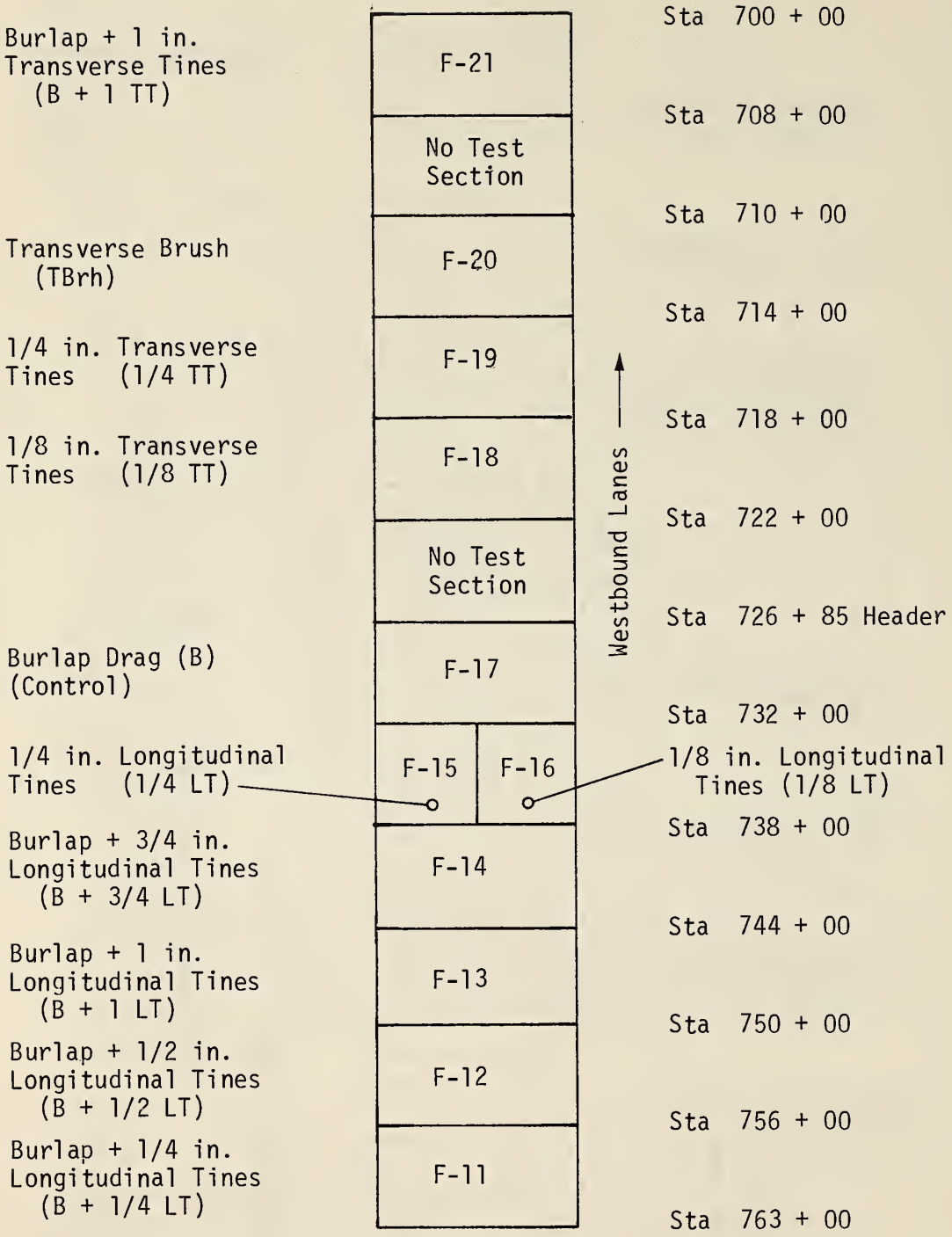


Figure 49. Location of test sections, IH 10 near Van Horn, Texas.

Table 9. Skid measurements under simulated rainfall on SH 6 in College Station, Texas, August 1972. (ASTM E274 with 14 in. tire). (50)
(continued next page)

Test Section and Description	Bald Tire WD (in.)	Bald Tire 20 (mph.)	Bald Tire 40 (mph.)	Bald Tire 60 (mph.)	Treaded Tire WD (in.)	Treaded Tire 20 (mph.)	Treaded Tire 40 (mph.)	Treaded Tire 60 (mph.)
F-1 Transverse Broom TXD = 0.043 in.	-0.020	64	40	28	-0.020	69	56	41
	0.004	67	36	23	0.004	75	54	37
	0.024	57	39	24	0.024	67	51	33
	0.050	58	28	16	0.050	74	55	25
F-2 Transverse Tines TXD = 0.064 in.	-0.059	72	48	36	-0.059	77	59	45
	0.035	77	47	32	-0.035	82	62	47
	0.000	72	56	43	0.000	74	62	49
	0.024	84	47	28	0.024	84	59	34
F-3 Longitudinal Broom TXD = 0.028 in.	-0.008	53	32	19	-0.008	71	49	33
	0.020	61	29	22	0.020	75	53	28
	0.031	54	29	19	0.031	71	51	33
	0.042	55	26	19	0.042	75	53	36
	0.051	66	28	21	0.051	74	51	27
	0.061	58	25	19	0.061	75	62	29
F-4 Longitudinal Tines TXD = 0.062 in.	-0.026	75	50	29	-0.026	80	61	35
	0.008	76	47	23	0.008	84	67	28
	0.028	69	35	20	0.028	75	64	28
	0.059	74	30	18	0.059	75	63	24

Table 9. Skid measurements under simulated rainfall on SH 6 in College Station, Texas, August 1972. (ASTM E274 with 14 in. tire) (continued)

Test Section and Description	Bald Tire WD (in.)	Bald Tire 20 (mph.)	Bald Tire 40 (mph.)	Bald Tire 60 (mph.)	Treaded Tire WD (in.)	Treaded Tire 20 (mph.)	Treaded Tire 40 (mph.)	Treaded Tire 60 (mph.)
F-5 Burlap + Longitudinal Tines TXD = 0.065 in.	-0.014	69	40	23	-0.034	67	59	42
	0.012	68	42	20	-0.019	72	58	34
	0.051	72	46	20	0.051	73	56	23
	0.071	74	46	18	0.071	76	55	18
F-6 Burlap Drag (Control) TXD = 0.032 in.	-0.010	46	26	16	-0.010	57	41	26
	0.004	50	28	16	0.004	61	45	21
	0.016	54	26	16	0.016	62	41	26
	0.055	60	31	16	0.055	64	47	20
F-7 Transverse Natural Brush TXD = 0.033 in.	0.000	64	35	25	0.000	83	63	43
	0.020	56	30	25	0.020	78	55	38
	0.031	66	32	23	0.031	85	65	36
	0.050	49	29	23	0.050	70	56	26

Metric Conversion: 1 in. = 25.4 mm
1 mph. = 1.609 km/h

Table 10. Skid measurements under simulated rainfall on IH 10 near Van Horn, Texas, October 1973 (ASTM E274 with 14 in. tire). (50)
(continued next page)

Test section and description	Bald tire WD (in.)	Bald tire 20 (mph)	Bald tire 40 (mph)	Bald tire 60 (mph)	Treaded tire WD (in.)	Treaded tire 20 (mph)	Treaded tire 40 (mph)	Treaded tire 60 (mph)
F-11	-.053	78	51	16	-.020	82	66	23
Burlap + 1/4 in. longitudinal tines	-.046	77	56	26	-.016	81	66	15
TXD = 0.081 in.	-.007	78	46	12	.019	81	62	17
	.065	80	52	9	.081	81	61	9
F-12	-.007	79	46	18	-.013	80	61	34
Burlap + 1/2 in. longitudinal tines	.002	81	45	14	-.009	79	59	16
TXD = 0.75 in.	.029	78	40	7	.011	83	61	14
	.063	78	38	3	.052	79	59	9
F-13	-.012	66	33	13	-.016	71	52	18
Burlap + 1 in. longitudinal tines	.064	60	32	11	.010	69	51	18
TXD = 0.062 in.	.014	70	30	5	.024	70	48	8
	.042	68	27	4	.050	71	46	8
F-14	.014	73	33	11	.024	81	63	14
Burlap + 3/4 in. longitudinal tines	.027	74	29	6	.035	78	54	8
TXD = 0.065	.052	75	33	6	.056	78	50	7
	.053	69	22	3	.072	82	60	11
F-16	.018	75	43	13	.013	81	66	25
1/8 in. longitudinal tines	.022	75	36	4	.023	82	65	17
TXD = 0.068 in.	.045	79	44	11	.040	84	65	20
	.069	77	42	4	.065	85	68	10

Table 10. Skid measurements under simulated rainfall on IH 10 near Van Horn, Texas, October 1973 (ASTM E274 with 14 in. tire). (continued)

Test section and description	Bald tire WD (in.)	Bald tire 20 (mph)	Bald tire 40 (mph)	Bald tire 60 (mph)	Treaded tire WD (mph)	Treaded tire 20 (mph)	Treaded tire 40 (mph)	Treaded tire 60 (mph)
F-17	-.011	65	31	10	-.009	77	57	16
Burlap drag (control)	-.002	62	25	4	-.003	71	52	16
TXD = 0.034 in.	.032	67	28	8	.030	78	49	14
	.053	65	21	4	.065	77	48	9
F-18	-.006	73	33	21	.022	88	65	22
1/8 in. transverse tines	.057	73	25	7	.054	94	65	9
TXD = .050 in.	.0002	79	32	20	.009	89	65	28
	.005	78	34	15	.016	88	65	17
F-19	.074	65	23	9	.086	79	55	8
1/4 in. transverse tines	.054	63	21	9	.056	78	61	22
TXD = .031 in.	.019	61	24	13	.019	85	64	18
	.006	60	25	14	.006	79	65	35
F-20	.045	59	20	5	.045	76	51	16
transverse brush	.039	60	20	5	.039	74	49	15
TXD = .021 in.	.032	56	24	10	.032	85	56	19
	.005	58	29	16	.005	85	62	31
F-21	.044	67	22	10	.044	77	55	14
Burlap + 1 in. transverse tines	.046	67	19	7	.046	75	50	9
TXD = .030 in.	.013	61	28	13	.012	73	49	16
	.006	62	23	11	.005	78	60	21

Metric Conversion: 1 in. = 25.4 mm
1 mph = 1.609 km/h

where

C_1, C_2, C_3, C_4, C_5, C_6 = constants developed from Equation 37.

A complete description of this technique is given in references 51 and 52. Examples of the equations developed for specific tire conditions follow:

East Bypass Equations

Transverse Textured Pavements

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{91}{V^{0.72}} \left[(13.75\ (TD + 1)^{0.13})\ TXD^{0.37} + \frac{1}{(WD + 0.1)^{0.25}} \right] \quad (39)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{124}{V^{0.74}} \left[(10.83\ (TD + 1)^{0.14})\ TXD^{0.41} + \frac{1}{(WD + 0.1)^{0.29}} \right] \quad (40)$$

Longitudinally Textured Pavements

ASTM-14" 24 psi (165.5 kPa)

$$SN = \frac{179}{V^{0.93}} \left[(8.4\ \ (TD + 1)^{0.13})\ TXD^{0.22} + \frac{1}{(WD + 0.1)^{0.16}} \right] \quad (41)$$

ASTM-14" 32 psi (220.6 kPa)

$$SN = \frac{229}{V^{0.89}} \left[(6.52\ (TD + 1)^{0.14}\ )\ TXD^{0.28} + \frac{1}{(WD + 0.1)^{0.11}} \right] \quad (42)$$

In an effort to provide more general equations, tire pressure was recognized as a relatively noninfluential parameter by noting the similarity of Equations 39 and 40 and of Equations 41 and 42. This simplification led to the equations shown in Table 11. Equations 43 and 45 represent the first step in a two step procedure and gave acceptable correlation coefficients, if somewhat high standard errors. The second step, resulting in equations 44 and 46 achieved a significant reduction in the standard errors, even though they remain somewhat high.

Table 11. Regression models for skid numbers.

Striation Direction	Equation Number	Regression Model	No. of Points	R^2*	SE**
Transverse	43	$SN = \frac{e^{7.8437}(TD+1)^{0.1799}TXD^{0.3306}}{V^{1.1455}(WD+0.1)^{0.3707}}$	168	0.76	17.0
Transverse	44	$SN = \left[\frac{239}{V^{1.15}} \quad 26.65(TD+1)^{0.18}TXD^{0.49} + \frac{1}{(WD+0.1)^{0.53}}\right]$	168	0.78	12.4
Longitudinal	45	$SN = \frac{e^{7.6994}(TD+1)^{0.1444}TXD^{0.0304}}{V^{1.3659}(WD+0.1)^{0.3266}}$	168	0.70	22.4
Longitudinal	46	$SN = \left[\frac{909}{V^{1.37}} \quad 3.22(TD+1)^{0.14}TXD^{0.06} + \frac{1}{(WD+0.1)^{0.31}}\right]$	168	0.76	13.6

*Coefficient of determination (correlation coefficient squared)

**Standard error in terms of skid number

The reason for these high standard errors is due to the data scatter present and the many variables - such as type of finish, location of test section, and type of aggregate - which were lumped together in this analysis. The effect of lumping these variables together can be seen in Figures 50 and 51 which are plots of predicted skid number versus observed skid number for each model. The data scatter can be seen, with the transverse texture model (44) exhibiting a higher degree of correlation than the longitudinal texture model (46). The lines of equality shown in Figures 50 and 51 are not regression lines and should not be interpreted as such. They do show that there may be other models which would more accurately represent the data.

It is true that fitting in a transformed range can introduce a bias in the model. This bias is clearly shown in Figure 50 and less clearly shown in Figure 51. The predicted SN can be corrected further by determining an equation between predicted and observed SN values which will take the observable bias out of the predictions, and possibly give even better correlation coefficients and standard errors.

The two step procedure used in this analysis cannot guarantee that the model which is finally derived is the best possible model, but it can arrive at relationships that are physically reasonable and statistically significant. Equations 43 through 46 are far better equations than could be found with any of the standard regression methods (linear, log, polynomial). The one additional step of removing bias in the residuals as shown in Figures 50 and 51 can and should be undertaken as a third step in a better representation of this data. These models for the prediction of skid number must be used with judgment as considerable variability between predicted values and actual values may exist. Keeping this in mind, models 44 and 46 were solved for several variables to illustrate the interaction of these variables.

The effect of speed on skid values as a function of texture direction and water depth is shown in Figure 52. Here the influence of texture direction is shown. At higher speeds (greater than 40 mph (64 km/h)) transverse texturing results in significantly higher values of skid numbers than longitudinal texturing. In addition, the surprisingly small influence of water depth is shown. The effect of vehicle speed on skid values as a function of texture direction and texture depth is shown in Figure 53. Here again, even for the very deep textured pavement (.060 in. (1.5 mm) in this example), transverse texturing results in significantly higher skid numbers at higher speeds where hydroplaning is most likely. Furthermore, deeper transverse textures do result in higher skid values at every speed.

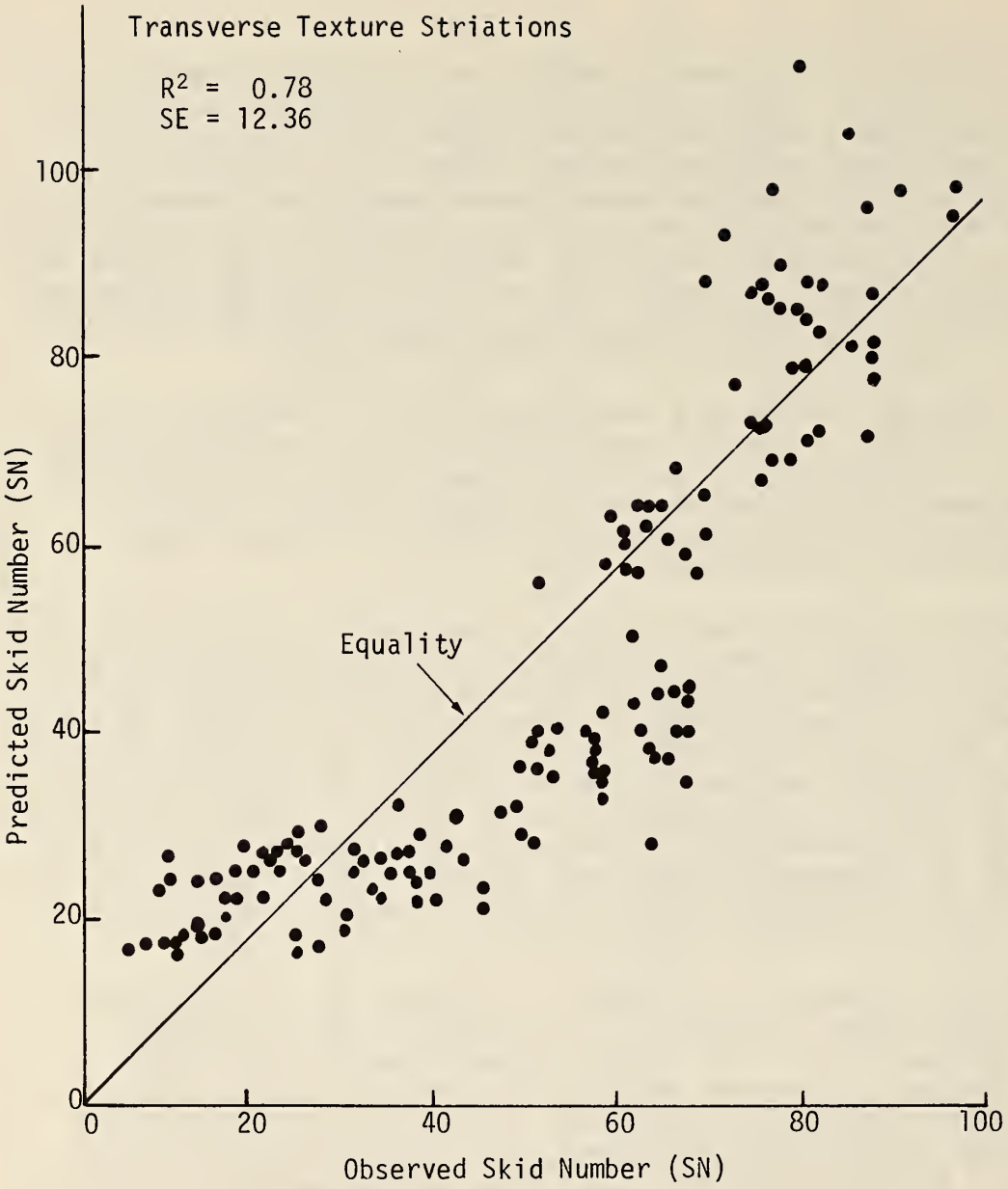


Figure 50. Predicted vs. observed skid number for transverse textures using model No. 44.

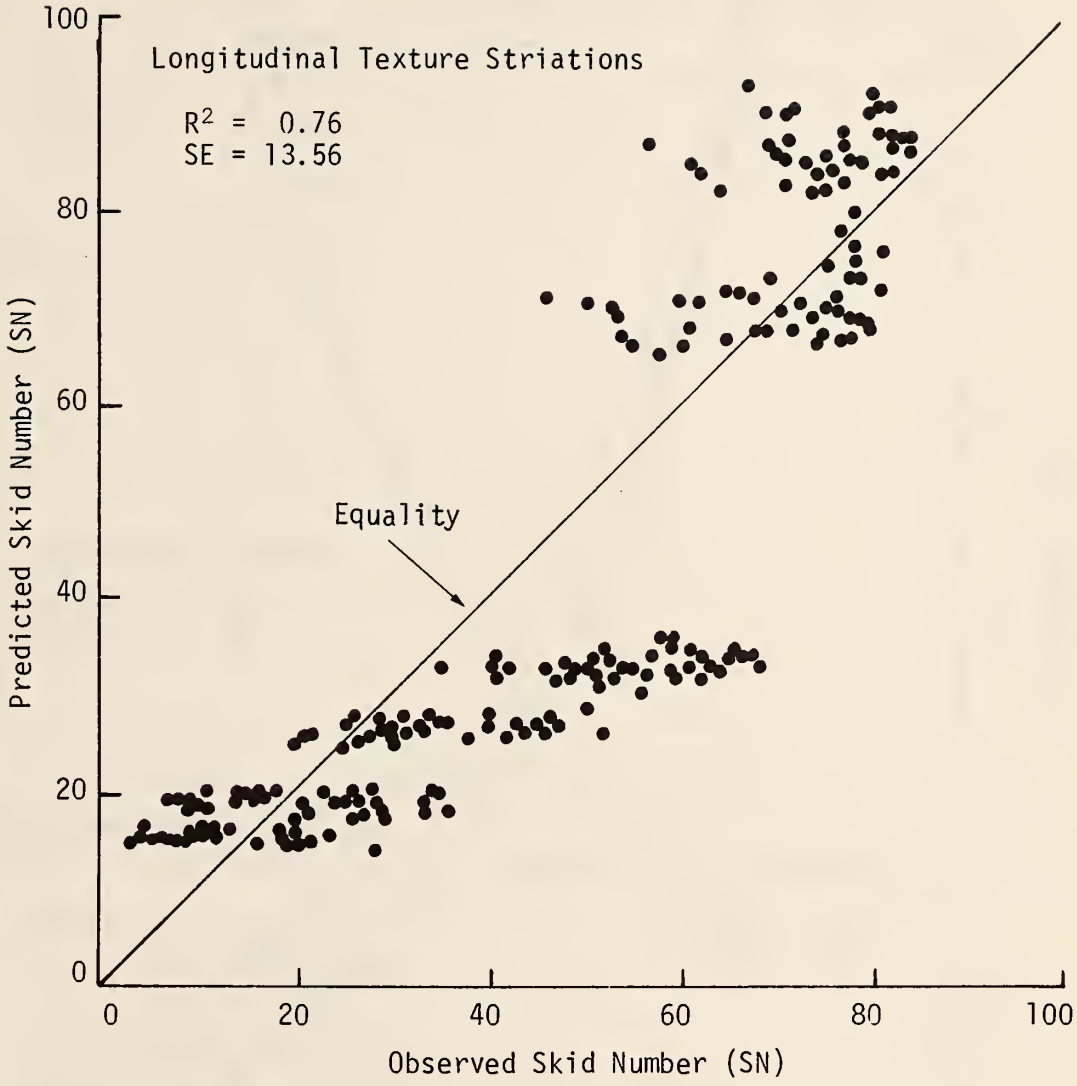


Figure 51. Predicted vs. observed skid number for longitudinal textures using model No. 46.

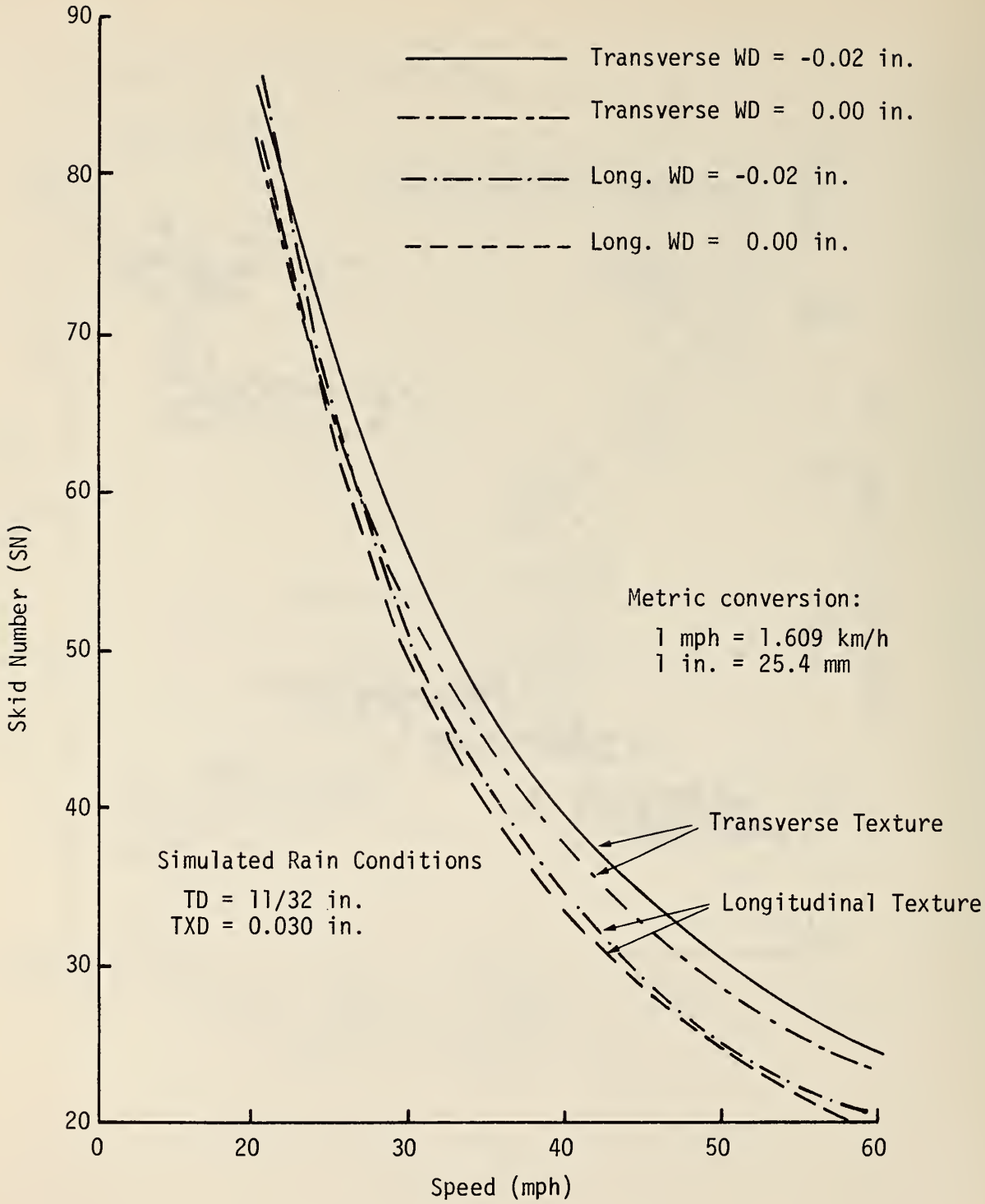


Figure 52. Effect of vehicle speed on skid number for tread depth of 11/32 in. and texture depth of 0.030 in. (curves calculated from equations)

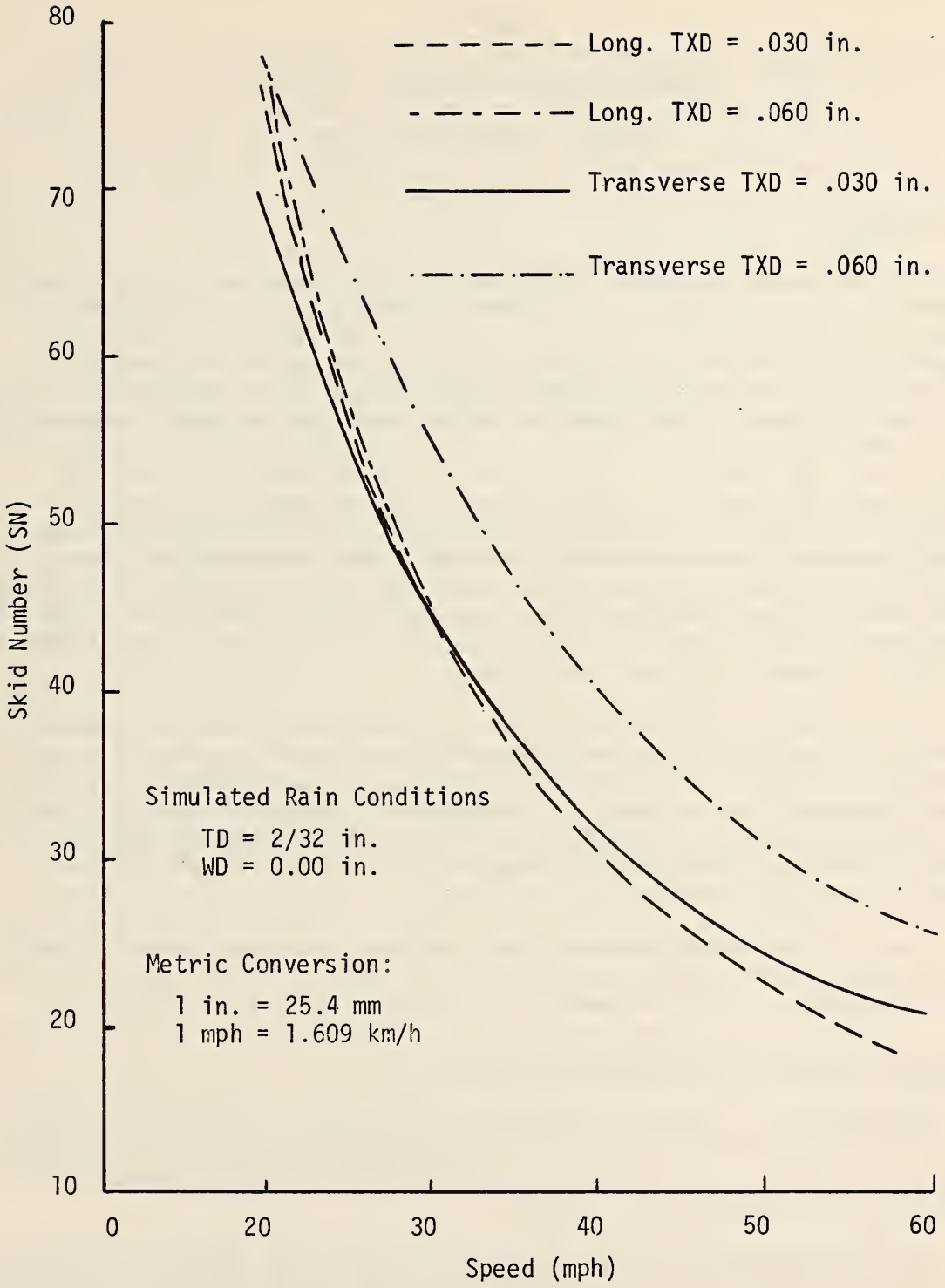


Figure 53. Effect of vehicle speed on skid number for tread depth of 2/32 in. and water depth of 0.00 in. (curves calculated from equations).

CHAPTER V

PAVEMENT CROSS-SLOPE CRITERIA AS RELATED TO VEHICLE CONTROL

Introduction

The 1965 edition of the AASHO (now AASHTO) Blue Book (13) states, "Important characteristics of (pavement) surface types in relation to geometric design are the ability of a surface to retain its shape and dimensions, the ability to drain, and the effect on driver behavior". Drainage can be improved by increasing the cross slope, but the effect of relatively steep cross slopes on driver behavior and vehicle control may be unacceptable. The Blue Book states that "cross slopes of 1/4 in. per ft. to 1 in. per ft. (2% to 8%) are noticeable in steering. The latter rate requires a conscious effort in steering and would increase the proneness to lateral skidding when vehicles brake on icy or wet pavements, and even on dry pavements when stops are made under emergency conditions." A survey of state practices conducted in 1972 (53) showed that the maximum cross slope used for either flexible or rigid pavement was 1/4 in. per ft. (2%). The survey included rural highways and urban freeways, but not city streets.

In the opinion of the researchers, further studies were needed and warranted to substantiate the AASHO guidelines regarding cross slopes, especially in light of recent technological advances concerning methods of studying vehicle handling. In particular, the researchers refer to the Highway-Vehicle-Object-Simulation-Model (HVOSM) (54, 55, 56), a computer program which simulates the dynamic interaction between the automobile and the roadway.

The object of this phase of the study was to conduct a preliminary investigation with HVOSM to determine the effects of cross slopes on both driver demands and vehicle control.

Research Approach and Evaluation Criteria

The HVOSM was used to simulate an automobile performing two common types of maneuvers. These were (1) travel with constant velocity along a tangent section and (2) a lane change maneuver at 60 mph (97 km/h) involving the traversal of a crown.

Certain factors, which are available through the HVOSM output, were used as indicators of the demand on both the driver and on the automobile during these maneuvers. Aligning torque on the front wheels of the automobile provided a measure of driver demands, and the required tire-pavement friction coefficient provided a measure of vehicle demands.

Aligning torques, which result from tire-pavement interaction forces, are dependent on properties of the tire, the vehicle, and the pavement. However, it must be pointed out that in HVOSM aligning torques are simulated by means of a constant "pneumatic trail" dimension only, i.e., the aligning torque is computed by simply multiplying the side force by the constant pneumatic trail dimension (an input variable). A correction is also made for gyroscopic precession. The degree to which HVOSM simulates actual aligning torques is therefore subject to some question, although qualitative comparisons between the torques produced on different cross slopes can still be made.

An automobile traveling along a tangent section with a cross slope tends to turn down the slope, and front wheel torques must be applied through the steering wheel to maintain a tangent path. The aligning torque to maintain a tangent path increases as the cross slope increases.

Aligning torques on the front wheels provide an indication of what is required of the driver as he performs a given steering maneuver. The relationships between aligning torques on the front wheels and the corresponding torque which must be applied at the steering wheel is dependent on a number of variables, such as type and condition of vehicle and type of power steering (if any). A review of the literature revealed very little information which identified this relationship, especially for automobiles with power steering. To gain insight into this area, the researchers performed some limited static tests on two automobiles (one had power steering). For the auto with power steering, it appeared that the steering wheel torque was practically constant and independent of the aligning torque. The power steering unit apparently had a compensating feature which adjusted the applied steering torque as a function of the aligning torque. On the car with no power steering, steering wheel torques were found to be proportional to the aligning torques. The steering wheel torque to aligning torque ratio was found to be approximately equal to the ratio of the wheel steer angle to the steering wheel angle.

To determine the tire-pavement friction coefficient necessary to perform the simulated maneuvers, the resultant of the tire side and circumferential forces developed during the maneuver was divided by the vertical tire force. This was done for each of the four tires, and then the one requiring the largest coefficient was selected. It is assumed that friction demands on any tire which exceeds available friction constitutes an unacceptable condition.

A large skid number for the tire-pavement combination was used in the simulation to insure an *availability of friction* during the maneuvers. Implicit is the assumption that the response of a vehicle during a given maneuver is independent of the *available friction* (as defined by skid number) provided the *friction demand* does not exceed the *available friction*.. This assumption is believed to be reasonable if the steer angle remains small, as was the case in most of the simulated maneuvers.

Parameters Investigated

Due to the limited scope of Phase I, it was necessary to limit the number of parameters investigated in the HVOSM studies as follows:

1. All runs were made with a standard size automobile, with a set of typical tires.
2. Only one type of pavement surface was studied (as characterized by a tire-pavement friction coefficient).
3. It was assumed that the cross slope was not interrupted by geometric irregularities such as ruts or local indentation or bumps. The problems associated with the retention of pavement shape and dimension are addressed elsewhere in this report.

The parametric study consisted of 16 HVOSM simulations. Eight cross slopes were investigated, beginning at 1/8 in. per ft. (1%) up to 1 in. per ft. (8%) in 1/8 in. (3.18 mm) increments. For each of these eight cross slopes, two basic vehicle maneuvers were investigated. These consisted of a 60 mph (97 km/h) constant velocity path along a tangent section and a 12 ft. (3.66 m) lane change maneuver at 60 mph (97 km/h) across a crown (cross slope on each side of crown equal in magnitude but in opposite directions). The maneuvers and the cross slope geometry are illustrated in Figure 54. The runs and the corresponding cross slope geometry are summarized in Table 12.

HVOSM

The computer runs were made with the Highway Safety Research Institute's (HSRI) version of HVOSM. This version of HVOSM is essentially the same as the "V-7" version (56), with the exception of a vehicle controller subroutine which HSRI added (57).

Researchers at HSRI developed the vehicle controller to simulate maneuvers which were very similar to those conducted in this study. These included curve traversals and lane change maneuvers. Although the HSRI researchers did not attempt to model or approximate human driver behavior with the controller, it is believed to be at least a gross representation of driver response for the maneuvers simulated, with one exception (explained in the following section).

Tangent Path Results

The required aligning torque versus time for run No. 1 is shown in Figure 55. Similar plots for runs 2 through 8 are given in Appendix A. Note that each plot is characterized by initial variations in the aligning torque (between time equal to zero and time equal to about 2.0 seconds). The variations become more pronounced as the cross slope increases. However, with the exception of the larger cross slopes, these

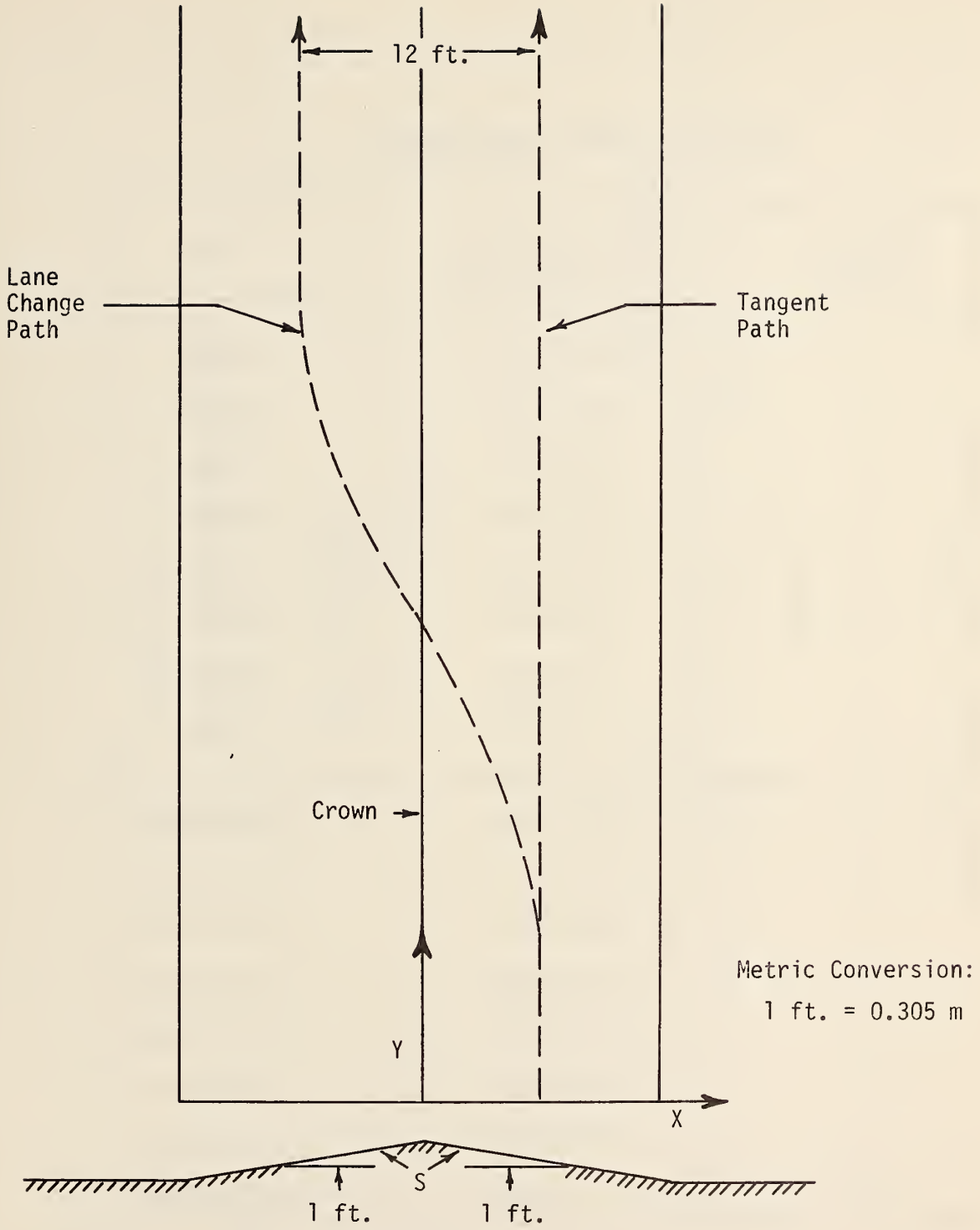


Figure 54. Maneuvers and cross slope geometry.

Table 12. Cross slope geometry.

Computer Run No.	Cross Slope in./ft., S*	Path
1	1/8 (1%)	Tangent
2	1/4 (2%)	Tangent
3	3/8 (3%)	Tangent
4	1/2 (4%)	Tangent
5	5/8 (5%)	Tangent
6	3/4 (6%)	Tangent
7	7/8 (7%)	Tangent
8	1 (8%)	Tangent
9	1/8 (1%)	Lane Change
10	1/4 (2%)	Lane Change
11	3/8 (3%)	Lane Change
12	1/2 (4%)	Lane Change
13	5/8 (5%)	Lane Change
14	3/4 (6%)	Lane Change
15	7/8 (7%)	Lane Change
16	1 (8%)	Lane Change

*See Figure 54.

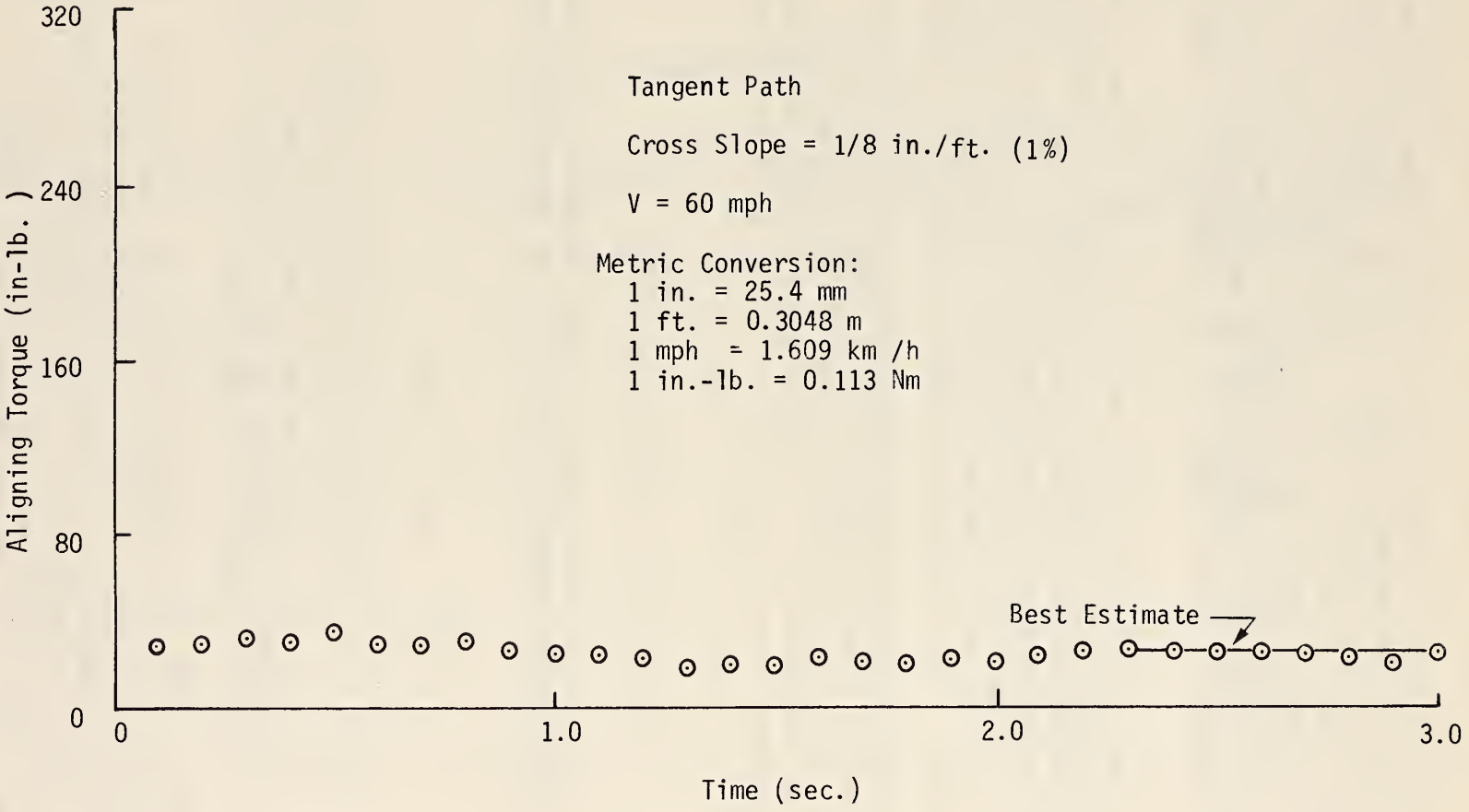


Figure 55. Run No. 1 aligning torque.

variations are transient in nature, and the aligning torque converges to a steady-state value as time increases. The transient response is caused by inexact initial conditions input to HVOSM. It is difficult to predetermine what steer angle, vehicle yaw, roll, etc., should be input as initial conditions. The "best estimate" line on each plot represents what is considered to be the steady-state aligning torque, i.e., the aligning torque which is required to maintain a tangent path on the given cross slope. A summary of the required aligning torques for the eight runs is shown in Table 13.

It can be seen that a steady-state value of aligning torque in runs 7 and 8 was not reached within the period of simulation (3.0 seconds), nor did it appear that the results were converging on a value. Changes to the damping characteristics of the controller did not appear to improve convergence. It is not clear, therefore, whether this result is controller oriented or is in fact how a driver may actually respond on relatively steep side slopes. Further studies are needed to clarify this observation.

As an illustration of how the aligning torque could be related to driver requirements, assume that one is driving along a tangent on a 3/8 in. per ft. (3%) cross slope at 60 mph (97 km/h) without power steering and the auto had a 25 to 1 steering wheel to steer angle ratio. If the steering wheel torque, T_{SW}, is 1/25 of the aligning torque (see Table 13), then

$$T_{SW} = \frac{1}{25} (96) = 3.84 \text{ in.-lb. } (.43\text{Nm})$$

For an 18 in. (457.2 mm) diameter steering wheel, the driver would therefore have to apply a total tangential force, F_{SW}, to the steering wheel, computed as follows:

$$F_{SW} = \frac{3.84}{18.0} = 0.2 \text{ lb. } (.89 \text{ N})$$

For a 3/4 in. per ft. (6%) cross slope, F_{SW} would equal 0.5 lbs (2.22 N) and for a 1/8 in. per ft. (1%) cross slope it would be 0.06 lbs. (.27 N), or the 3/4 in. per ft. (6%) cross slope would require about eight times more "effort". The extent to which these driver "requirements" approach, or exceed, driver "limits" remains to be determined.

Also shown in Table 13 is the required friction coefficient to maintain a tangent path at 60 mph (97 km/h) for the eight cross slopes. It is interesting that in each case the rear tires demanded the greatest friction to maintain the tangent path. Also, the largest part of the demand of the rear tires is for traction, i.e., the circumferential tire force (approximately 100 lbs. (445 N) per tire) necessary to maintain the 60 mph (97 km/h) speed. Since the tractional demands did not increase

Table 13. Summary of tangent path results.

Run No.	Cross Slope (in./ft.)	Required Aligning Torque (in.-lb.)	Required Friction Coefficient	Required Friction Coefficient for Static Equilibrium
1	1/8 (1%)	25	0.092	0.010
2	1/4 (2%)	60	0.097	0.021
3	3/8 (3%)	96	0.088	0.031
4	1/2 (4%)	128	0.098	0.042
5	5/8 (5%)	164	0.110	0.052
6	3/4 (6%)	204	0.110	0.063
7	7/8 (7%)	240	0.110	0.073
8	1 (8%)	260	0.120	0.083

Metric Conversion: 1 in. = 25.4 mm
1 ft. = 0.305 m
1 in.-lb. = 0.113 Nm

appreciably with cross slope, the required friction coefficient did not increase appreciably. Also shown in Table 13 is the friction coefficient needed to keep a stationary vehicle from sliding off the cross slope. The value of the coefficient in such a case is simply the cross slope in radians.

Information in Table 13 is shown plotted in Figure 56. The results of these simulations indicate that increases in cross slope do not appreciably increase frictional demands of the automobile for travel along a tangent path. However, demands of the driver increase considerably as the cross slope increases.

Lane Change Results

The results of the lane change maneuver for the 1/8 in. per ft. (1%) cross slope are shown in Figure 57. Similar plots for lane change maneuvers on other cross slopes are given in Appendix B.

Three quantities were plotted for each maneuver, namely, the required friction coefficient, the vehicle's path, and the aligning torque. These quantities are plotted as a function of the longitudinal distance, or the distance along the roadway.

Note in Figure 57 that the "required friction coefficient" plot and the "aligning torque" plot are characterized by large initial values. As the lane change maneuver starts, the aligning torque suddenly jumps from a very low value to 1300 in.-lb. (146.9 Nm). These large values for the aligning torque and the required friction coefficient are a direct result of the way in which the controller initiates the lane change. At the time specified for the lane change maneuver to begin, the controller suddenly inputs a relatively large steer angle to the simulated vehicle. For example, in run No. 9, the steer angle jumped from practically no steer angle to 3.8 degrees within 0.005 seconds at the beginning of the lane change. This would represent a steering wheel turn of approximately 100 degrees. Such steering inputs are obviously not representative of a typical driver. Unsuccessful attempts were made to "ramp" the steer input at a more reasonable rate by adjusting controller damping parameters. This is not to say, however, that the controller cannot be modified to ramp the steer input at the beginning of a lane change maneuver. Such modifications are recommended as part of the Phase II effort to better simulate the typical lane change maneuver. Although the steering input during the initial part of each lane change maneuver was unreasonable, the steering input as determined by the controller during the remainder of each lane change appeared reasonable.

It can be seen in Figure 57, and the figures in Appendix B, that friction demands and aligning torques decrease after the high initial values. They then increase again as the vehicle is steered back in the other direction toward a tangent path on the opposite side of the crown.

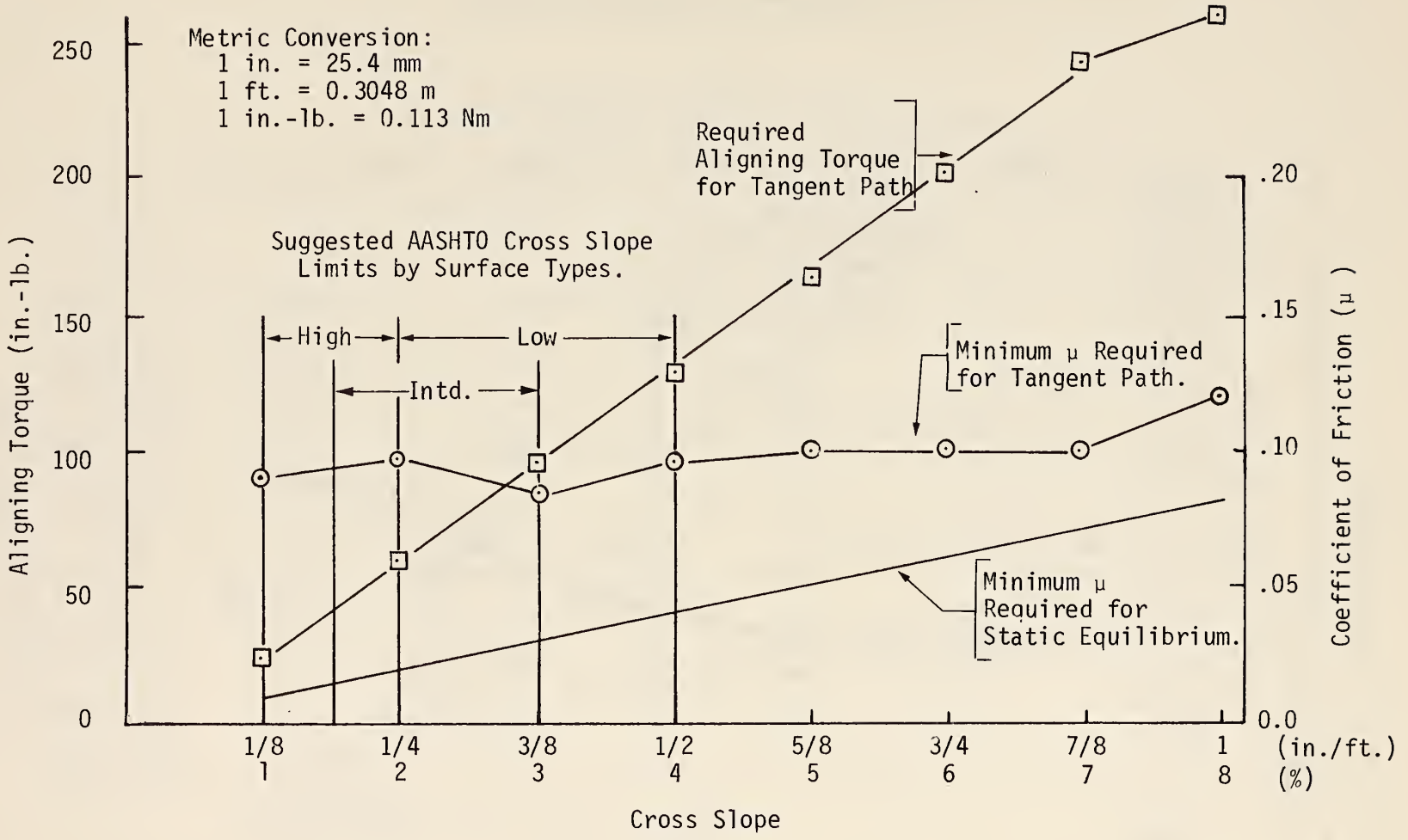


Figure 56. Tangent path results.

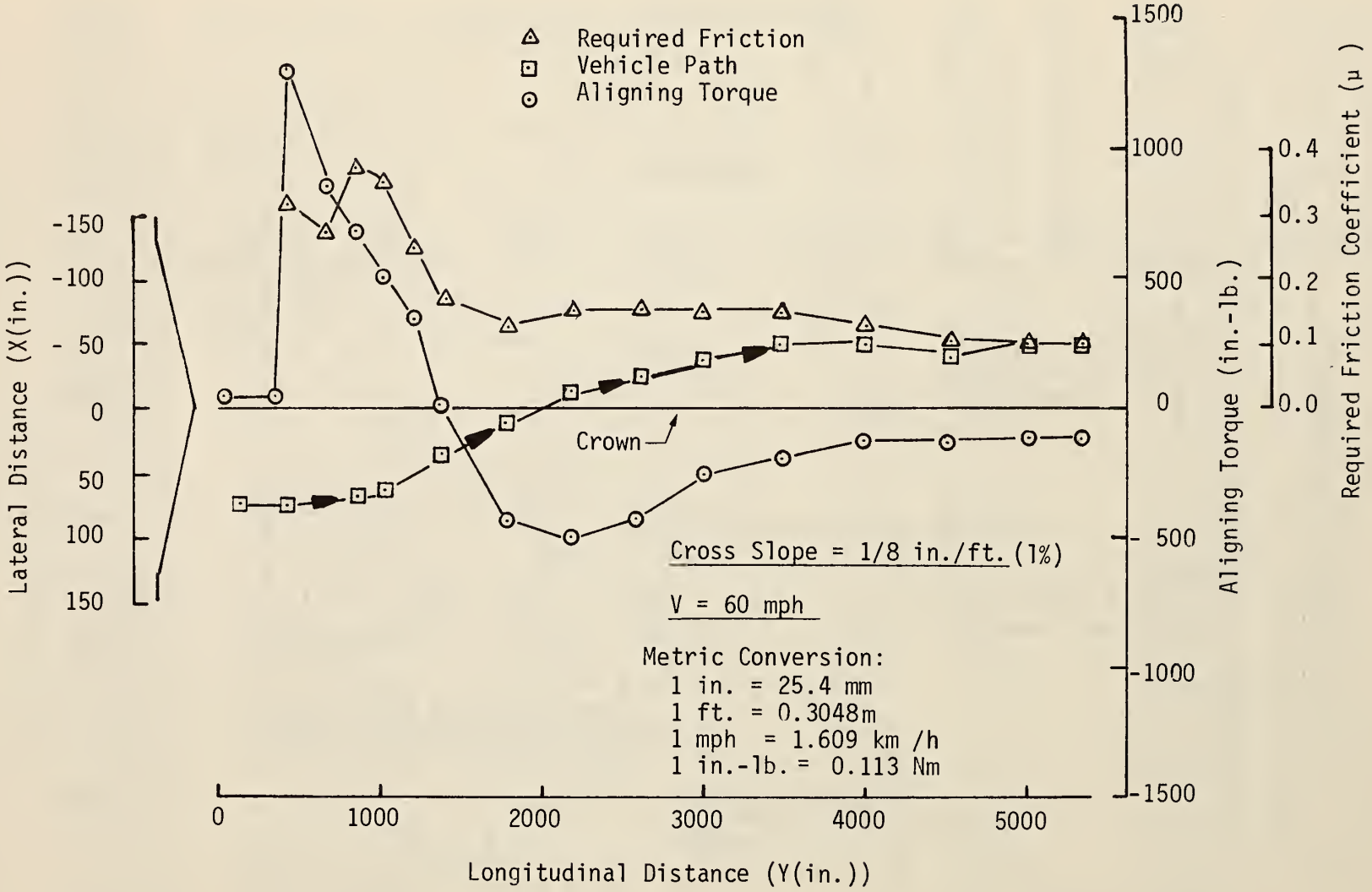


Figure 57. Run No. 9 results.

Table 14. Summary of lane change maneuvers.

Run No.	Cross Slope (in./ft.)	Required Aligning Torque (in.-lb.)	Required Friction Coefficient
9	1/8 (1%)	475	0.16
10	1/4 (2%)	550	0.18
11	3/8 (3%)	635	0.22
12	1/2 (4%)	720	0.20
13	5/8 (5%)	840	0.25
14	3/4 (6%)	1000	0.31
15	7/8 (7%)	1300	0.28
16	1 (8%)	1440	0.34

Metric Conversion: 1 in. = 25.4 mm
1 ft. = 0.305 m
1 in.-lb. = 0.113 Nm

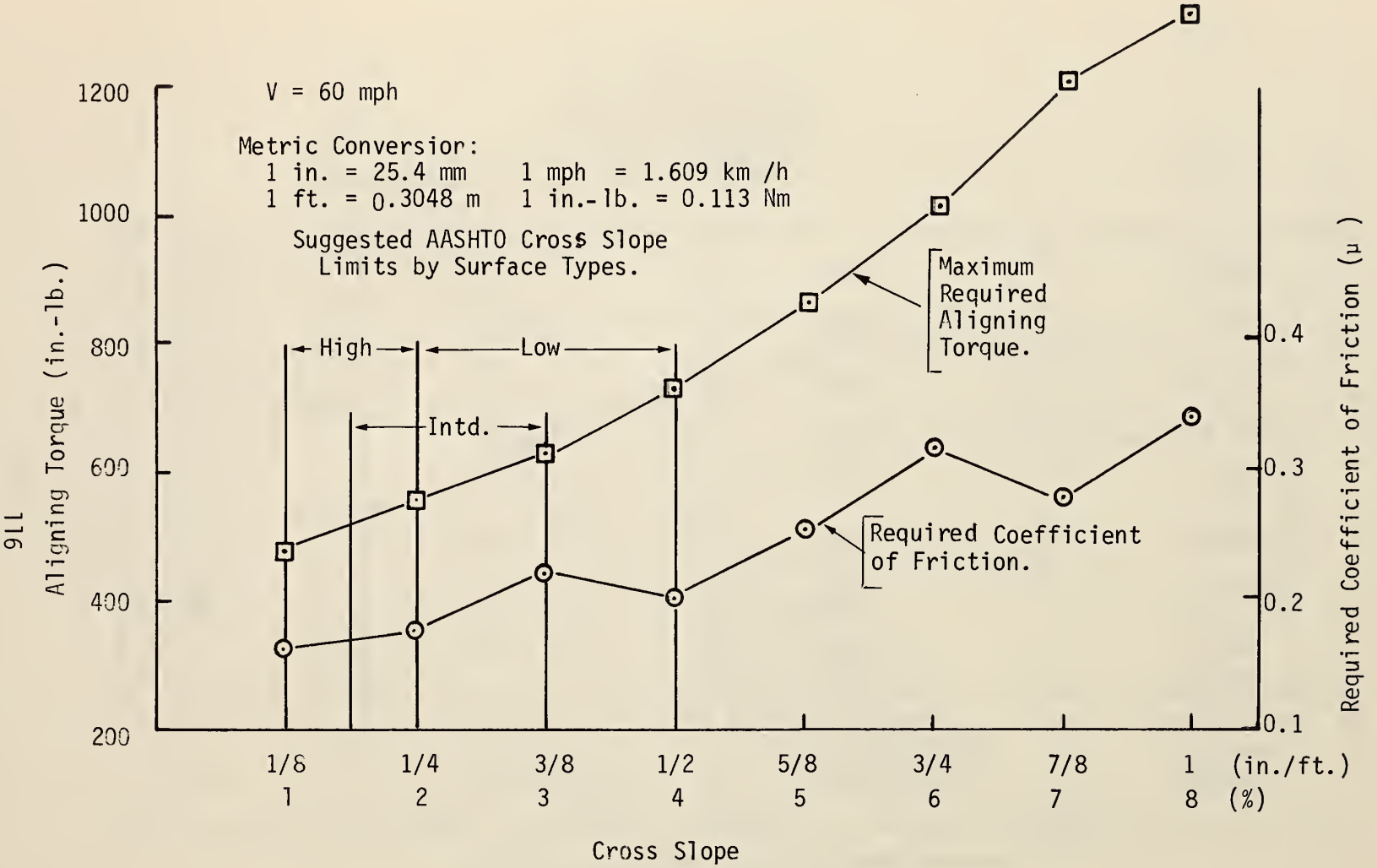


Figure 58. Aligning torque and required coefficient of friction vs. cross slope for lane change.

It is noted that the rear tires demand the greatest friction during most of the lane change maneuver, as was the case in the tangent path maneuvers. This is due to the combined tractional force and side force requirements of the rear tires.

Maximum values of the required friction coefficient and the aligning torque are shown in Table 14 and are plotted in Figure 58. These values were not obtained from the initial part of each curve since the steering input there is considered unrealistic. They were obtained from the second peak of each curve, or that part of the maneuver where the vehicle is being steered back toward a tangent path. It is assumed that demands in this part of the maneuver are as great as they are during the initial phases of the maneuver.

In general, the maximum friction demand during the lane change appears to be proportional to the magnitude of the side slope. There are intermediate peak values of the friction demand, which occur at cross slopes of 3/8 in. per ft. (3%) and at 3/4 in. per ft. (6%). It is not known if these cross slopes do in fact require more friction to negotiate than steeper slopes, as shown in Figure 58, or if the results for these cross slopes represent "scatter" in the HVOSM data. It should be noted that the controller routines which steer the vehicle and which apply traction for speed control continually sense deviations in the desired path and vehicle speed. However, deviations do occur which require corrections. Although attempts are made in the control routines to minimize the number and magnitude of these corrections, oscillations in the desired path and speed sometimes occur. The degree to which corrections are necessary is a function of the initial conditions of the vehicle (speed and orientation), the vehicle's characteristics, the roadway geometry, and the controller algorithms themselves.

A comparison of Figures 56 and 58 shows that for a given cross slope the lane change maneuver requires a significantly greater driver effort (aligning torque) than does driving a tangent path. Vehicle demands (tire-pavement friction coefficient) of the lane change are also considerably greater than those for the tangent path. However, with regard to driver effort, since most travel time is spent driving tangents, it seems reasonable to assume that considerably larger driver efforts would be tolerable for short periods of time, such as that needed to make the lane change. An analogous situation cannot be made with regard to friction demands. A very undesirable condition exists anytime the required friction exceeds the available friction. In other words, the friction demands of the lane change maneuver must be met.

Friction demands for a constant speed tangent path appear practically independent of the magnitude of the cross slope. The largest demand comes from the rear tires due to the combined tractional force and side force requirements. However, increases in cross slope require increases in the steer angle of the front wheels which would undoubtedly increase tire wear. Studies are needed to quantify the wear as a function of cross slope.

In both the tangent path and the lane change maneuver, the vehicle remained stable and there were no indications that loss of control was impending. However, for the larger cross slopes, some erratic responses occurred. As can be observed in the plotted results, the required friction and aligning torques did not reach a steady state condition for either the tangent path or the lane change (after the lane change was completed).

Conclusions

It is concluded that the HVOSM computer program can be used to quantify the effects of cross slope on both driver demands and vehicle control. However, before further investigations with the HVOSM are made, modifications are needed with regard to the vehicle controller routines. Such changes would result in a better simulation of a typical driver's response in performing various functions, such as the lane change maneuver and in maintaining a constant vehicle speed. Modifications may also be warranted to the HVOSM with regard to the simulation of aligning torques on the tires.

Tentative conclusions which can be drawn from the limited parametric study are as follows:

1. An automobile can maintain a tangent path on cross slopes up to about 3/4 in./ft. (6%) without significant friction demands (as measured by tire-pavement friction coefficient) or driver effort (as measured by aligning torques). For greater cross slopes, the simulation exhibits an erratic response which is probably a numerical problem related to control inputs.

2. The lane change maneuver (including crossing a crown) requires considerably larger friction demands and driver effort than does driving a tangent path. While the larger driver effort may be tolerable for the lane change (since it occurs over a relatively short period of time), the friction demands must be met, or unacceptable consequences will result.

3. Friction demands of approximately 0.10 for a constant speed tangent path appear practically independent of the magnitude of the cross slopes. The largest demand comes from the rear tires due to the combined tractional force and side force requirements. However, increases in cross slope require increases in the steer angle of the front wheels which would undoubtedly increase tire wear.

4. Required driver effort has been quantified in terms of aligning torques on the front wheels. More study is needed to (a) relate aligning torques to actual driver requirements (steering wheel torque) and (b) to determine human tolerance "limits" in terms of steering wheel torques.

5. Cross slope values up to 1/4 in. per ft. (2%) show no significant detrimental effects with regard to either friction demand or driver effort.

CHAPTER VI

A DETERMINATION OF DEFICIENCIES IN EXISTING SURFACE DRAINAGE DESIGN METHODOLOGY FOR SAG VERTICAL CURVES

Introduction

Current design methods to accomplish surface drainage for sag vertical curves appear to have evolved from the drainage design of tangent sections. At first glance, this procedure would appear to be adequate, however, this is only due to the infrequent occurrence of a "substantial" amount of surface drainage runoff. Modern highway design practice calls for a high standard of safety for the highway user and a maximum utilization of all sections of highway transportation facilities. However, drainage of sag vertical curves remains a troublesome problem with respect to both safety and facility utilization.

The presence of water in the travel lane produces several potentially hazardous situations. It provides a means to reduce safety by reducing the available skid resistance of the pavement (11, 6). The presence of a heavy water film is an essential element in hydroplaning (58). Also, visibility is reduced due to the water spray of preceding vehicles. A literature review revealed that an increased hazard results from improper drainage of wide, flat pavement sections (59) and at superelevated sites (59, 53). No literature was uncovered which dealt specifically with surface drainage of sag vertical curve sections; the hazard produced by wet pavements increases due to the increased period of time that surface water exists on the travel lanes. Surface drainage paths on sag vertical curve sections are necessarily longer, thus necessitating a longer time interval than would occur on flat, tangent sections. This is due to the buildup of drainage runoff from the decreasing change in absolute longitudinal grade and also to the collection of runoff from both directions at the apex of the curve. This indicates that the probability of an accident increases because of the additional time increment necessary to adequately drain the pavement surface of a sag vertical curve. Therefore, the presence of a sag vertical curve alignment must be taken into consideration in the overall design of surface drainage.

To determine any deficiencies in existing surface drainage design methodology for sag vertical curves, an examination of the critical design elements for surface drainage of sag vertical curves and the current state and national design policies has been conducted. These data and the findings of the library research are the basis of material reported herein.

Design Elements Considered

Several design elements have been identified as relating to surface drainage of sag vertical curves. A policy dealing with some or all of

these elements would be beneficial. The design methodology of the surface drainage of sag vertical curves should include some of these factors as they relate to the specific climatic, terrain conditions and roadway type. Any design policy must be flexible enough to accommodate these conditions.

As with drainage design of all roadway sections, the drainage area must be determined in order to estimate the needed capacity of the drainage system. From the contour of the sag vertical curve section of pavement, the lines of flow should be established. The length and width of each drainage basin should be analyzed. The total pavement width to be drained in one direction is a design element necessary to determine drainage areas. This is in relation to the distance from the curb face or drainage ditch hinge point of the crest of the cross slope of the pavement.

The drainage area of curbed pavements should be analyzed for each individual section requiring a curb inlet. Normal spacing of inlets on tangent sections vary from 300 to 500 feet (91.4 to 152.4 m) as a maximum. Within the sag vertical curve section, the drainage area doubles within the vicinity of the low point due to drainage from both grades. Any overflow from other drainage inlets will accumulate at the lowest point thus requiring an increased design at the low point.

As defined previously, the elements essential to design predictions of rainfall are the rainfall intensity based on the climatic conditions of the site and the increment of duration of this activity. The storm recurrence interval used to predict the maximum design intensity of sag vertical curve will depend on the safety factor associated with the importance of the roadway, but could be justifiably greater than a normal tangent section due to the runoff accumulation at the low point within the curve.

Curb capacity of the drainage system is controlled by several design elements. Batter height and shoulder and gutter cross slope control the cross section of the collected water in combination with the longitudinal grade of the gutter. For operational safety, the width of encroachment of the collected water should be controlled.

For the analysis of uncurbed pavements, the drainage system for not only the pavement area but also the slope faces adjacent to the drainage channel should be examined. The hydraulic capacity of the drainage system downstream from the site should be considered and channel designs should reflect this. The cross-section dimensions, the runoff coefficient and the drainage grade will control the capacity of the channel or side ditch.

Prior to collection, storm water flows in sheets across the pavement toward the shoulders. The depth of this water film increases along a line of flow having a length proportional to the grade and cross slope (11). The change in grade from negative to positive within many sag

vertical curves complicates the flow length; however, as long as the flow continues to travel within the traveled way, the increasing depths create an undesirable hazard. The grade at the low point of curve approaches zero, thus appearing to have the length of the lines of flow equivalent to the pavement width from the point of the crown. However, the change of grade occurring on either side of the low point necessitates the interception of several lines of flow at the lowest edge point. The depth of the water film at the extreme ends of these flow lines should be controlled by proper design even though the mechanics of depth calculations for non-constant grades have yet to be determined. To insure the complete removal of the storm water from the traveled way, the transition between any two surfaces of varying textures should be designed to allow continuous flow across the interface. Specifically, the pavement-shoulder or the lane-paved shoulder interface should be examined.

The hydraulic capacity of storm water inlets depends upon the flow characteristics of the curb and gutter system and the geometry of the grating, if applicable. Inlet efficiency is dependent on the depth of flow above the inlet opening, but quite often diminishes due to plugging by debris. Safe designs of inlet capacities should include the possibility of partial plugging.

Based on the survey of the states and the literature review, the following areas of possible design standards need to be examined:

Drainage Area
Rainfall Duration and Intensity
Drainage Channel Capacity for Uncurbed Sections
Length of Flow Path
Cross Slope at the Low Point
Texture Variation of Roadway Shoulders
Hydraulic Capacity of Curb and Gutter Sections
Hydraulic Capacity of Drainage Inlets

Examination of Design Policies

AASHTO National Policy - The governing design standards of modern highways vary considerably among various engineering organizations. This is most certainly true of standards associated with sag vertical curve alignment. Much of the dispersal of modernized design policy has been provided by AASHO's "A Policy on Geometric Design of Rural Highways" (13). It is believed, however, that specific policy toward the surface design methodology for sag vertical curves is deficient in the current Blue Book.

Basically, the Blue Book sets four different criteria for establishing the length of sag vertical curves. The criteria established for use in current practice includes (1) headlight sight distance, (2) rider

comfort, (3) drainage control, and (4) a rule of thumb for general appearance (13). For overall safety on highways, AASHO reported that a minimum length of sag vertical curve should be based on a headlight sight distance equivalent to the stopping sight distance for the design speed. The criterion is expressed in terms of the "K" rate for the range of grade differences allowable. The design variable "K" is equal to the length of the curve (L) divided by the algebraic differences between the two grades (A). Consideration of comfort requires that the vehicular rate of change of grade does not exceed 1 ft./sec.^{2} (.305 m/s^2). The length of vertical curve associated with this criterion is normally well below that necessary to provide the needed length of curve for headlight sight distances. As a rule of thumb, the Blue Book suggests a minimum length of 100 x A as a design aid for general appearances of alignments consisting of small or intermediate values of A.

The single criterionaffecting surface drainage is related only to curbed sections of all roadway types. A separate grade line of not less than 0.35 percent for the outer edges of the pavement is recommended for the last 50 ft. of both grades prior to the level point. For other than firm subgrades, a grade line of not less than 0.5% is required. The minimum grade of 0.35% corresponds to a "K" value of 143. No drainage criteria for uncurbed pavements are included. Inlet placing in the gutter line of a sag vertical curve is considered, but the AASHO recommendation does not appear to be sufficient. The general consideration of inlet spacing on curbed roadways states that one inlet should be placed at the low point of the curve, and one inlet on each side of the low point where the grade elevation is approximately 0.2 ft. (61 mm) higher. This criterion is rather general since it does not take into consideration the actual grade of the curve, the cross slope, the inlet capacity, the length of runoff flow, drainage pavement width, nor design rainfall intensity.

The capacities and locations of all drainage structures as noted in the Blue Book

> "...should be adequate to minimize damage to upstream property, to prevent the saturation of the roadbed and to secure as low a degree or risk of traffic interruption by flooding as is consistent with the importance of the road and the design traffic volume."

Drainage structures should also provide the necessary storage space on the pavement surface such that collected water does not create an unsafe condition of excess water depth within the traveled way by encroachment. Along with these, a need exists in the complete removal of storm water from the traveled way.

State Design Policies - It should be recognized that AASHO policy provides a basic minimum from which progressively improved standards of various design agencies develop. To inquire as to the evolution of the surface drainage design of sag vertical curves, a questionnaire was

developed. This questionnaire was designed to aid in the interpretation of each respective state's design policy toward surface drainage of sag vertical curves. The majority of the states indicated that AASHO standards prevailed in their state, thus confirming the evolution process of this design over a period from 1965 to the present date.

In April 1974, questionnaires were sent to the chief design engineer of nine states selected on the basis of their probable contributions to the project and to provide a broad geographic data base. Three immediate responses were received. The remaining six completed questionnaires were received in early June after a follow-up letter was mailed in the hope of initiating other responses. The follow-up letter included another set of the same questionnaire anticipating misplaced originals. A presentation of the data collected in this survey is provided in Table 15.

Summary of Responses to Sag Vertical Curve Drainage Design Questionnaire

For purposes of analyzing the responses to the questionnaire, a point by point summary was found to be most beneficial.

1. *Has a wet pavement speed limit been in existence in the past or is one being contemplated for the future?* _____*Yes* _____*No*
   If yes, what speed? _____*mph*

No wet pavement speed limits were found to exist in the responding states. Two states noted the existence of a state law requiring a "safe" speed under the warranting conditions of the roadway.

2. *In your opinion, to what extent is an investigation of pavement surface drainage incorporated into the total geometric and cross section design of your state's highways?*
   _____*Insignificant*
   _____*Significant*
   _____*Highly Significant*

Seven states indicated that a significant investigation of pavement surface drainage is being incorporated into the total geometric and cross section design of highways in their respective states. One state replied that it was highly significant for curb and gutter sections while insignificant for non-curb and gutter sections. One insignificant response was recorded.

3. *Is pavement surface drainage especially designed for the presence of sag vertical curves?*
   *Yes*_____*Special consideration*
   *Yes*_____*Some consideration*
   *No*_____*No more than any other alignment*

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire. (continued)

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
1. Wet pavement speed limit (yes or no)	no	no[1]	no	no[1]	no	no	no[1]	no	no
2. Significant investigation of pavement surface drainage (yes or no)	yes	yes	yes	yes	yes	no[2]	yes	yes	no
3. Consideration of the presence of sag vertical curve for pavement surface drainage (some, special, or no)	some	some	spec	no	no	no[2]	no	spec	no
4. Average storm recurrence design interval (yes or no)	yes	--	yes	no	no	yes	yes	yes	yes
a. Interval used or suggested (yr.)	8-15	--	8-15	1	--	1-3	3-8	3-15	8-15
b. Corresponding rainfall rate (in./hr.) (mm/h)	var.	--	6-7 (152-178)	?	--	3 (76)	4 (102)	4.7 (119)	2.2 (56)

[1]Noted the existence of a state law requiring a "safe" speed under the warranting conditions of the roadway.

[2]Differentiates between curbed and non-curbed sections. An affirmative response was given for curbed sections.

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire. (continued)

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
5. Variation of storm interval among various roadway elements (yes or no)	yes	--	yes	--	--	yes	yes	yes	yes
a. Two-way drainage level grade (yr./in./hr.) (yr./mm/h)	--	--	10/6-7 (10/152-178)	--	--	--	-/4 (-/102)	10/4.7 (10/119)	10/2.2 (10/56)
b. Sag vertical curve with steep grades (yr./in./hr.) (yr./mm/h)	10/-	--	10/6-7 (10/152-178)	--	--	--	-/4 (-/102)	50/6.0 (50/152)	50/3.1 (50/79)
c. Superelevated horizontal curve (yr./in./hr.) (yr./mm/h)	10/-	--	10/6-7 (10/152-178)	--	--	--	-/4 (-/102)	10/4.7 (10/119)	10/2.2 (10/56)
d. Sag vertical underpass (yr./in./hr.) (yr./mm/h)	50/-	--	25/6-7[3] (25/152-178)	--	--	50/-	25/-	50/6.0 (50/152)	50/3.2 (50/81)
e. Intersection of sag vertical curve and crossroad (yr./in./hr.) (yr./mm/h)	10/-	--	10/6-7 (10/152-178)	--	--	--	-/4 (-/102)	10/4.7 (10/119)	50/3.1 (50/79)
f. One-way drainage, multi-lane & gore width > 36 ft. (yr./in./hr.) (yr./mm/h)	--	--	10/6-7 (10/152-178)	--	--	--	-/4 (-/102)	10/4.7 (10/119)	--

[3]Storm recurrence interval for a sag vertical curved underpass varied with roadway type: Interstate - 50 years; Primary highway - 25 years; Secondary highway - 10 years, approximately 6-9 in./hr. (152-229 mm/h).

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire. (continued)

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
6. Maximum water film depth used as a criterion (yes or no)	no	no	no	no	no	no	no	no	no
a. Criterion specified (in.)	--	--	--	--	--	--	--	--	--
b. Recommended maximum depth (in.)	.25	0.03	--	0.0	--	--	.25	--	--
(mm)	(6.3)	(0.8)		(0)			(6.3)		
c. Depth measured from top of macro-asperities (yes or no)	yes	yes	--	yes	--	--	yes	--	--
7. Maximum length of flow path uses as a criterion (yes or no)	no	--	yes	no	no	no	no	no	yes
a. Criterion used (ft.)	--	--	900	--	--	--	--	--	300
(m)			(22.9)						(7.6)
8. Maximum pavement width of one-way surface drainage (yes or no)	yes	--	yes	no	yes	yes	no	yes	yes
a. Recommended width (ft.)	36	--	36	--	24	--	--	36	36
(m)	(11)		(11)		(7.3)			(11)	(11)

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire. (continued)

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
9. Increased cross slope for extreme right lane for drainage of wide pavements (yes or no)	yes	--	yes	yes	no	no	no	yes	no
10. Applicable criteria governing curbing with sag vertical curves (yes or no)									
a. Design traffic volume	no	--	yes	yes	no	no	no	no	no
b. Shoulder erosion	yes	--	yes	no	yes	yes	yes	no	yes
c. Longitudinal grade	yes	--	yes	no	no	no	no	no	no
d. Annual rainfall	no	--	yes	no	no	no	no	no	no
e. Pavement width	yes	--	yes	no	no	yes	no	no	no
f. Other	no	--	yes[4]	yes[4]	no	no	yes[5]	yes[4]	no
g. Drainage area	yes	--	yes	no	no	no	no	no	no
h. Design speed	no	--	yes	yes	no	no	no	no	no
i. Cross slope	yes	--	yes	no	no	no	no	no	no
j. Pavement texture	no	--	no	no	no	no	no	no	no
k. Superelevation	no	--	yes	no	no	yes	no	no	no
11. Curb criteria vary among roadway types (yes or no)	no	--	no	yes	no	no	no	no	no

[4]Right-of-way width may determine use of curbing or its non-use.

[5]Rural versus urban section of roadway.

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire. (continued)

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
12. Cross slope criteria for sag vertical curves (%)									
a. Freeway Minimum	1.56	--	2.0	1.56	1.5	2.08	2.1	1	2.0
Maximum	8.0	--	8.0	2.08	---	Var.	9.0	6	2.0
b. Expressway Minimum	1.56	--	2.0	1.56	1.5	7.08	2.1	1	2.0
Maximum	8.0	--	8.0	2.08	---	Var.	9.0	6	2.0
c. Two lane Minimum	1.56	--	2.0	1.56	1.5	7.08	2.1	1	2.0
Maximum	8.0	--		2.6	---	Var.	9.0	6	2.0
13. Features incorporated into intersection drainage design									
a. Total pavement width drains or gutters (yes or no)	no	--	no	no	no	none	no	no	no
b. Diversion of all surface water onto minor roadway (yes or no)	yes	--	no	yes	no	none	no	no	no
c. Steeper cross slopes	yes	--	yes	no	no		no	no	no
d. Open-graded surface mix	no	--	no	no	no		no	no	no
e. Increased storm recurrence interval	yes	--	yes	no	no		no	yes	yes

Table 15. Tabular summary of responses to the sag vertical curve drainage design questionnaire.

Questions	Responses Made by Each State								
	1	2	3	4	5	6	7	8	9
13. continued									
f. Transverse grooving	--	--	no	no	yes		no	no	no
Crown side roads	--	--	--	--	yes		--	--	--
Check contour for flat spots	--	--	--	--	---		yes	--	--
14. Features incorporated into surface pavement drainage of underpasses (responses)									
-Increased storm interval	--	--	yes	--	--	yes	--	yes	--
-High capacity catch basin	--	--	--	--	yes	--	--	yes	--
-Pumping station - 25 yr. design	--	--	--	--	--	--	yes	--	--
-Limit ponding to half the outside lane	--	--	--	--	--	--	--	yes	--

Three states responded "yes" with "some consideration" of sag vertical curves being used in the design of pavement surface drainage. One state's negative response concerning its policy noted the use of a maximum "K" value of 143 as described in AASHO throughout all sections of roadway. Two other states also replied "no more than any other alignment." The final state again differentiated between curb and non-curb sections by designing especially for sag vertical curb and gutter sections.

4. *Is an average storm recurrence design interval used for the surface pavement drainage design? _____Yes _____No*
Please indicate the interval used, or in your opinion, a desirable storm recurrence design interval.
< 1 year _____ 1-3 years _____ 3-8 years _____
8-15 years_____ 15-25 years_____ 25-50 years_____
> 50 years_____
What is the corresponding rainfall rate?_____in./hr.

5. *Is the same average storm recurrence design interval for surface pavement drainage used for all of the roadway elements listed below? _____Yes _____No*
If Yes, please neglect the remainder of this question; if no, please indicate the interval used for each of the following applicable elements of roadway design.
(a) Two-way drainage, level grade ____years____in./hr.
(b) Sag vertical curve with both grades >3° ____years____in./hr.
(c) Superelevated horizontal curve ____years____in./hr.
(d) Sag vertical underpass ____years____in./hr.
(e) Intersection of sag vertical curve & crossroad ____years____in./hr.
(f) One-way drainage, multi-lane + gore width >36 ft. ____years____in./hr.

In response to questions 4 and 5, three states responded "no" to the use of a storm recurrence design interval. All of these states had noted in their introductory letter that they had no design criteria specifically for sag vertical curves other than the standards contained in AASHO publications.

Six states have attempted to use a storm recurrence design interval in relation to surface pavement drainage. Eight to fifteen year intervals were reported as policy corresponding to a varying rainfall rate throughout the different state locales. The average intensity rates reported ranged from 2 in./hr. (50.8 mm/h) to a maximum of 7 in./hr. (177.8 mm/h). For most elements of roadway design, the average storm recurrence interval was used with the exception of the policy toward underpasses utilizing a sag vertical curve. A considerably longer recurrence interval was reported as policy by these states toward underpass design. Twenty-five and fifty year intervals with varying corresponding rainfall rates from 6 to 9 in./hr. (152.4 to 228.6 mm/h) were reported by the six

states. Two states also included sag vertical curve sites, along with underpass sections, for the design interval increase. No variation among the policies governing the three basic types of roadway was reported with the exception of one element of design in one state. It is the policy of one state to vary the interval for the design of a sag vertical curve underpass. The policy in that state on the design intervals for Interstate highways, primary highways, and secondary highways was 50 years, 25 years, and 10 years respectively.

6. *Is a minimum water film depth of pavement surface water used as a design criterion?* _____*Yes* _____*No*
If so, what criterion is specified? _____*in.*
If not, in your opinion, what maximum water film depth would be desirable to minimize the slipperiness of the wet pavement? _____*in.*
Is the water film depth indicated above measured from the top or the bottom of the macro-asperities of the pavement?
_____*Top*
_____*Bottom*
_____*Other means of defining depth*
*Explain*_____
_____

All states reported that no criterion for a maximum water film depth of the pavement surface water in their design policy exist. However, four of the chief design engineers suggested a maximum desirable water film depth. Two of the engineers agreed on a 1/4 in. (6.4 mm) depth, whereas the other pair indicated a maximum depth of 0.03 and 0.00 in. (0.8 and 0.0 mm) respectively. These depths were indicated as having been measured from the "top" of the macro-asperities of the pavement. No other means of defining the water film depth was explained in the responses.

7. *Is a maximum length of the flow path of the runoff on the surface pavement a criterion in drainage design?* ____*Yes* ____*No*
If so, what is the criterion? _____*ft.*

Although most of the responses from the states were negative, two states did report that a maximum length of the flow path of the surface pavement drainage was a criterion of the policy used in designing sag vertical curves. The maximum was stated as being 900 ft. (274 m) on highways "with medians or swales" and 450 ft. (137 m) on highways "with shoulder drainage or curbs." The other states reported 300 ft. (91 m). One of the states responding "no" noted that "catch basins are usually placed a maximum of 300 ft. (91 m) apart, and closer on flat grades."

8. *For multi-lane highways, is a maximum pavement width of one-way surface pavement drainage recommended?* _____*Yes* _____*No*
If Yes, what is the maximum width? _____*ft.*

9. *In a multi-lane facility, is a steeper cross slope permissable for the extreme right lane?* _____*Yes* _____*No*

The responses related to the design policy of multi-lane facilities varied considerably from state to state. Six states replied that their respective design policies recommend a maximum pavement width of one-way surface pavement drainage. Four responses of 36 ft. (11 m) maximum width were provided. All of these states replied that their policy permitted a steeper cross slope for the extreme right lane of a multi-lane facility. The third affirmative reply indicated a maximum width of 24 ft. (7.3 m) with a noted exception of pavements wider than 48 ft. (14.6 m). In conjunction with this recommendation, this state's design policy did not permit a steeper cross slope for the extreme right lane.

10. *What criteria govern the use or nonuse of curbing in conjunction with sag vertical curves? (Circle all applicable criteria.)*

(a) Design traffic volume
(b) Shoulder erosion
(c) Longitudinal grade
(d) Annual rainfall
(e) Pavement width
*(f) Other. Explain*_____
(g) Drainage area
(h) Design speed
(i) Cross slope
(j) Pavement texture
(k) Superelevation

11. *Do the criteria governing the use or nonuse of curbing in conjunction with sag vertical curves vary among Freeway, Expressway, and Rural Two-Lane Highway design?* ____*Yes* ____*No*

The responses to these questions indicated that the criteria governing the use or nonuse of curbing in conjunction with sag vertical curves does not vary among freeway, expressway, and rural two-lane highway designs. The specific criteria governing curbing varied widely among the responding states with one state not completing the remainder of the questionnaire. All respondents indicated shoulder erosion as being a criteria of their respective highway design. Two responses indicated criteria common to the design policy of each as being longitudinal grade, pavement width, drainage area and cross slope. One of these states also indicated the applicability of design traffic volume, annual rainfall, design speed, and superelevation as criteria of the design of sag vertical curves. Frequent explanations of curbing criteria are right-of-way adequacy, and rural vs. urban highway section. Responses to the explanation of other governing criteria included "curbing is used in rural areas to control side slope erosion in high fills," and "nonuse for high volume, high-speed traffic conditions."

12. *What cross slope criteria are followed for a sag vertical curve on the following:*

Freeway (Divided)	*minimum*____%	*maximum*____%
Expressway (with partial or no access control)	*minimum*____%	*maximum*____%
Two-Lane	*minimum*____%	*maximum*____%

Cross slope criteria for sag vertical curves expressed in the responses did not vary within the respective states' policies concerning the design of freeways, expressways, and two-lane highways. The range of minimum cross slopes provided as responses was from 1.0% to 2.1%. The indicated range of the maximum cross slopes was between 6% and 9%. This reply could seem to indicate a misunderstanding of the second part of this question. It also could be reasoned as implying the use of superelevation within a sag vertical curve due to the combination type alignment with a horizontal curve of a magnitude necessitating maximum superelevation. One state did note "superelevation" with his response to a maximum cross slope.

13. *For intersection drainage design, what features are incorporated when the major road is on a sag vertical curve?*
_____*Total pavement width drains or gutters*
_____*Diversion of all surface water onto minor roadway*
_____*Steeper cross slopes*
_____*Open-graded surface mix*
_____*Increased storm recurrence interval*
_____*Transverse grooving*
_____*Other. Explain*_____

The questionnaire shows a wide variety of solutions that are being incorporated into the drainage design of intersections located on sag vertical curves. Two prevalent themes were detected among the responding states. One important factor in the intersection design deals with prohibiting drainage water from egressing onto the major road. Two states replied that pavement drainage which is present on the side road is being directed off onto the shoulders by means of crowning the side street, and placing catch basins on both minor sides of the intersections. Gutter lines and shoulder ditches are being designed by another state to prevent the occurrence of flat spots such that collected water will not flood back onto the major roadway. Another feature for adequate intersection drainage design as related in the responses deals with increasing the drainage ability of the major roadway in the vicinity of the intersection. Within the section of sag vertical curves consisting of relatively small grades, an increase of the pavement cross slope at or near the intersections is currently being recommended in the design policy of two states. Transverse grooving is also being considered as a design feature to increase the pavement drainage characteristics at or near intersections. The state indicating the use of transverse grooving noted that it was being used "experimentally to correct drainage and skidding at intersections." Those states that indicated the use of a storm recurrence interval as a design aid also indicated an increase of this interval, thus allowing for higher rainfall intensities in intersection design.

14. *What features does your state incorporate into the design of surface pavement drainage of underpasses with sag vertical curves?*_____

Elements of design incorporated into pavement drainage of underpasses with sag vertical curves varied widely among the responses. Design appears to be focused on the prevention of complete flooding of the "enclosed" section of roadway due to the sag vertical curve and overhead structure. Catch basins are used in one state in batteries to make sure water does not pond. Three states that indicated the use of a storm recurrence interval in design also indicated an increased interval for underpass design and considered providing more inlets than normally required. Another state provides a pumping station for some underpasses designed to handle a 25-year storm recurrence interval based on the locale within the state. This particular question was presented in an unstructured format in the hope of acquiring a spontaneous response from chief engineers. Apparently, a question with a format similar to question #13 elicited a response more readily. A reasonable response was provided to the request to explain other, more unique methodologies whereas the structured responses provided additional information.

In summary, the significant findings from the questionnaire survey are listed below:

1. No wet weather speed limits are currently in use.

2. Surface drainage is a significant design consideration in most states.

3. A majority of the states sampled did not give sag vertical curves any special consideration in regard to drainage.

4. A majority of the states sampled do use a storm recurrence interval as a basis for drainage design. The interval was 8-15 years. Underpasses did in general receive more attention.

5. None of the states indicated that a maximum water film depth was being used as a design criterion. Maximum depths from 0 to 1/4 in. (0 to 6.4 mm) above the top of the pavement macroasperities were suggested.

6. A maximum flow length is used as a criterion by only two of the reporting states.

7. Thirty-six ft. (11 m) maximum width of drainage in one direction is common.

8. The most common criteria governing curbing along sag vertical curve sections are the prevention of shoulder erosion and right-of-way availability. There seems to be no variable criteria among the various types of roadway.

9. Cross slope criteria for sag vertical curves were also typical of standard criteria for tangent sections.

10. Few attempts have been made to incorporate experimental drainage features at critical drainage locales. Those states whose policy prescribes the use of a storm recurrence design interval also encourage increasing the design interval for underpasses and intersection design for sag vertical curves.

Statement of Deficiencies

In general, it can be stated that current design methodology merely "allows" for the surface drainage of rain water and does not design "in" realistic provisions for the control, collection, and disposal of the expected runoff. Establishment of policy control of various design elements would alleviate a majority of the deficiencies of sag vertical curve design and improve the surface drainage of these roadway sections. A table of tentative criteria for sag vertical curve drainage design compiled from the survey of the states is presented in Table 16. Values or ranges of values are provided for various design elements which are justified herein.

As a minimum design, a maximum pavement width for one directional drainage should be established as policy throughout all sections of roadway. A majority of the states surveyed indicated a policy of a maximum pavement width of 36 ft. (11 m) would be desirable.

For curbed sections, the drainage area for each inlet should be controlled by a maximum spacing requirement. One state showed considerable work in this field and had set a reasonable design policy range between 300 to 500 ft. (91 to 152 m). In the vicinity of the low point of sag vertical curves, this policy should be examined more closely. A desirable design level could range from 50 to 200 ft. (15 to 61 m).

The analysis of the drainage characteristics of each specific sag vertical curve site was expressed as design policy of several states questioned in the survey. This policy should be encouraged throughout all design agencies. A storm recurrence design interval is reflective of this analysis. In reference to urban freeways and arterial streets, AASHO states (60):

> "It is appropriate to use storm frequencies of 10 to 100 years for design of storm sewer systems to insure against flooding of the highway or adjacent areas."

It is believed that similar design criteria could be adopted for surface drainage of highways. Based upon the responses from the questionnaire, an interval of approximately ten years is currently considered as a minimum drainage design interval for all sections of roadway. Considering the desire of several states to increase the storm recurrence design interval for critical drainage sections, it is felt that the ten-year interval would serve as a minimum design criteria and a longer interval, perhaps 25 years, for a desirable design interval. Sections with highly

Table 16. Tentative sag vertical curve drainage design criteria based on a survey of existing practice.

Design Element	Desirable Design	Minimum Design
Maximum pavement width for one way drainage	36 ft. (11 m)	36 to 48 ft. (11 to 14.6 m)
Maximum inlet spacing	50-200 ft. (15 to 61 m)	300 to 500 ft. (91 to 152 m)
Storm recurrence design interval with crossroad intersection with underpass structure	25 yr. 25-50 yr. 25-50 yr.	10 yr. 10-25 yr. 25 yr.
Channelized curb flow encroachment	Containing ponding on the shoulder.	Containing ponding on the shoulder and 1/2 of the extreme right lane.
Increase in Roughness Coefficient (Manning's N) for various materials due to small gutter slopes	0.005	0.002
Pavement cross slope for sag vertical curve section	Slight increase in cross slope	No increase in cross slope

critical drainage such as underpasses and crossroad intersections implementing sag vertical curves should also be designed with an increased recurrence design interval such as tentatively listed in Table 16. An investigation as to the relative values for both minimum design and desirable design criteria will be necessary. The corresponding rainfall intensities with these intervals are apparently adequate for the corresponding risk involved with the interruption of traffic for a duration of one hour. However, for operational safety, rains of much shorter duration are believed to contribute to hazardous situations. For sag vertical curves, the wet pavement problem is even more conducive to accidents. Therefore, the duration increment for surface drainage design of sag vertical curves should be appropriately reduced. Based on the minimum reporting interval of the U. S. Weather Bureau, a five-minute duration would probably prove to be the least practical design interval. In relation to the storm recurrence design interval, shorter duration interval storms should be investigated in more detail in order to establish a minimum design value.

Without concise computations of the water film depth within a sag vertical curve, caution should be taken in recommending design criteria along these lines for this type of section. No criteria for water depth were found among the states surveyed. Several of the engineers questioned recognized excess water depths as being a hazard to the motoring public. Criteria developed for tangent sections should be initiated for sag vertical curves with liberal safety factors to assure adequate coverage for the variation due to the changing grades.

The prevailing design criterion among the responses for placement of curbing along a sag vertical section of roadway was the erosion characteristics of the roadside slope. With new knowledge about constructing and testing of the subbases and subgrades in recent years, this criterion should be more carefully examined in the future prior to the construction of a curb and gutter system. The availability of right-of-way was also mentioned in the questionnaire responses. The economics of projects in certain areas will still require the use of a curb for channelizing the surface pavement runoff in many sag vertical curve locations. In order to promote safety, the encroachment of the channelized water along the curb and gutter should be minimized.

Maximum runoffs should be analyzed for the width of the flow and removed to an underground system when the flow widens to a point such that it hazardously encroaches into the travel lane. An investigation of curb and gutter capacities for maximum design rainfall would be in order to determine what criteria would best suit not only sag vertical curve sections but also tangent sections of roadway. Based on recommendations of one of the responding states, a minimum design criteria has been tentatively established for the encroachment of channelized curb flow throughout a sag vertical curve. For a minimum design criteria, the maximum encroachment of flow-out from the curb should be contained within the right half of the extreme right lane and the shoulder if any area is provided between the curb and the traveled way.

A closer examination of the wetted perimeter available for channelized curb flow would be beneficial to establish a desirable design criteria, but it would appear that the containment of runoff on the shoulder only would be desirable.

An increase in the value of the roughness coefficient (Manning's N) is based on the recommendation for small gutter slopes (61) as found at the low point of sag vertical curves. The increased roughness coefficient would more conservatively estimate the flow quantity as collected within the sag vertical curve.

Just as an increased cross slope is allowed in the extreme right lane on a multi-lane facility due to drainage problems of wide pavements, the cross slope should be increased in the vicinity of the low point. This criterion is subject to other findings dealing with pavement cross slope criteria.

From all evidence available, the procedure for designing the surface drainage of intersections within a sag vertical curve is quite diversified. General guidelines would greatly improve the design procedure such that no important step of the overall drainage system is overlooked or incomplete. As mentioned previously, a total analysis of the drainage system of each site is necessary, with an increased storm recurrence interval for specific locations such as crossroad intersections.

The same increased interval for the design analysis of underpass sag vertical curves applies. Those features related to drainage of underpasses as described in the questionnaire responses are applicable for specific sites, but the best overall criteria for flexibility would be an increased storm recurrence design interval for a total analysis.

The tentative criteria, as discussed previously, have been formulated from an interpretation of the responses of the survey of existing practices of sag vertical curve drainage design. Those values or range of values that were based on a judgmental basis from the recommendations of the states will necessitate further study to provide the necessary data to confirm or disconfirm these tentative criteria. Field studies in Phase II are proposed to provide the information necessary to draft a basic policy on the design of drainage facilities for sag vertical curves.

Recommendations of Possible Study Sites

In anticipation of analyzing highway sections with a sag vertical curve alignment, a request for appropriate study sites was made in the introductory letter accompanying the questionnaire. The letter requested a set of six sites having some degree of drainage problem. In order to obtain a representative sample of all types of

highway design, both sets were to be divided equally among three roadway types: freeways, expressways, and two-lane highways. In the introductory letter, a freeway was defined for purposes of this questionnaire as a divided highway with full access control. An expressway was defined as a divided highway with partial or no access control. No specific definition of the two-lane highway was made due to the clarity of the term.

Adequate space was provided for all 12 site descriptions in the "Site Recommendations Outline". The chief engineers were requested to give a detailed description of the recommended sites. Most of the states provided a map as requested, indicating the location of the described study site. A summary of the nine responses to the "Site Recommendations Outline" follows:

Summary of Site Recommendations

1. Descriptions and locations of eight sites of sag vertical curves were provided by the State of Kentucky. The four freeway and four expressway sites are all associated with some degree of drainage problems. All four expressway sites are located within ten miles of Lexington, Kentucky, on U. S. 60. Three of the freeway sites are in the northwest quadrant within a 50-mile (80.5 km) radius of Lexington on Interstates 64, 71, and 75. A map with these locations circled was furnished as were plans and profiles of the expressway sites.

2. A complete outline of 12 sites was provided by Pennsylvania. All three roadway types were recommended with and without any apparent drainage problem. No map was provided; however, it has been found that Harrisburg is encompassed by Cumberland County, York County and Dauphin County, the locations of at least one site of each of the three roadway sites with and without drainage problems.

3. Twelve sites within the State of Michigan were pinpointed on an accompanying map and described on the outline. No expressway sites existed; therefore, extra interstate freeways were included. Four of the freeway sites without drainage problems and three of the freeway sites with drainage problems are located in the vicinity of Detroit. Several of these sites in Detroit, both with and without drainage problems, are noted as being scheduled to be grooved in the third quarter of 1974. These sites could be examined in both the before and after stages. A detailed map of Detroit has each of these sites marked.

4. A complete outline of twelve sites was provided by the State of Colorado. Although the descriptions of each of the sites were short and incomplete, the detailed county maps that were provided offer excellent visual interpretation of the locations.

With the exception of one two-lane location, all sites recommended for Colorado are in the Denver-Boulder vicinity. All road types and both drainage types could be investigated outside Denver. No association was found between "sag vertical curves" and "high accident sites" with a probable reason being "...a relatively high frequency of wet pavement involvement..." in accidents.

5. A complete outline was provided by the State of Washington thus providing two alternatives for each of the six classifications. The Seattle-Tacoma area generated nine of the sites, most of the sites being located on various connecting highways between the two major cities. All of the freeway sites were located on Interstate 5. State Road 99 between Seattle and Tacoma was indicated as being a major expressway having sites with and without drainage problems along its route. Located just outside this major corridor in the Puget Sound area are sag vertical curve sites on various two-lane highways serving the outlying communities. Access to all sites appears to be excellent with respect to their location as pinpointed on the state highway map accompanying the questionnaire response.

6. The Texas Highway Department suggested study sites in two of the state's districts. Detailed maps were provided marking the various locations of the recommended sites. All sites were located in the coastal area in the vicinity of Houston and Beaumont, Texas. These coastal sites will prove to be of interest due to a minimum of grade composing the sag vertical curve alignment.

7. Along with the response to the questionnaire, the road design engineer of the State of Minnesota provided a description of ten recommended sites for further study. Three of the ten sites were noted as being past or present locations correlated with possible hydroplaning, although the alignment is not a sag vertical curve. Of the ten sites, six are located in the vicinity of St. Paul. Both drainage types of the two-lane highway and the freeway road types are included by the sample in and around St. Paul. A map of the state accompanied the Site Recommendation Outline.

To date, a total of 78 sites have been suggested by seven of the nine responding states. Reasonable access to 61 of these sites could be accomplished through major cities of the respective states. The major cities in the vicinity of the sag vertical curve sites are Denver, Colorado; Seattle, Washington; Houston, Texas; Harrisburg, Pennsylvania; Lexington, Kentucky; Detroit, Michigan; and St. Paul, Minnesota. These rural sites are usually located within a 40-mile (64.4 km) radius of the major city probably due to the preference of the state engineer. This will be beneficial to the time and cost of the Phase II study and would

still be anticipated as providing a representative sample. It is the opinion of the research team that valuable information could be acquired from the previously mentioned sites.

CHAPTER VII

SUMMARY OF TENTATIVE CRITERIA

TO REDUCE HYDROPLANING

Based on the information developed in preceding chapters a relatively clear definition has evolved concerning the influence of different factors on the full loss of control forces (dynamic hydroplaning). It is of interest now to estimate the range and distribution of these factors on the highway. Approximate distributions of these factors are given in Figure 59. The approximate 50 percentile values of these obviously skewed distributions are

$$TD = 7/32 \text{ in. } (5.6 \text{ mm})$$
$$P = 28 \text{ psi } (193 \text{ kPa})$$
$$TXD = 0.025 \text{ in. } (0.64 \text{ mm})$$

The lower five percentile level of the three groups is approximately

$$TD \leq 2/32 \text{ in. } (1.6 \text{ mm})$$
$$P \leq 21 \text{ psi } (145 \text{ kPa})$$
$$TXD \leq 0.01 \text{ in. } (0.25 \text{ mm})$$

Note that the frequency plot of texture depth is not the same as exposure rate to traffic since the more highly traveled, and thus more highly polished roads, are probably biased toward the lower texture range. Therefore the frequency plot given is probably more optimistic than warranted in terms of actual exposure to the lower texture values. The obvious conclusion is that significant numbers of automobiles are traveling under conditions which would produce hydroplaning at speeds less than the current speed limits of 55 mph. Obviously poor compliance with this speed limit further aggravates the situation.

However, there is one factor that makes the situation less critical than our initial considerations would seem to indicate and that is the comparative rarity of rainfalls intense enough to sustain significant positive water depths on the road surfaces.

In a recent report (64) climatological data were analyzed and presented to determine the comparative frequency of different intensity rainfalls. Figure 60 illustrates these probabilities for Central Texas. It seems apparent that the same techniques used for the Texas data could be used for all states, assuming the availability of appropriate climatological data for other states.

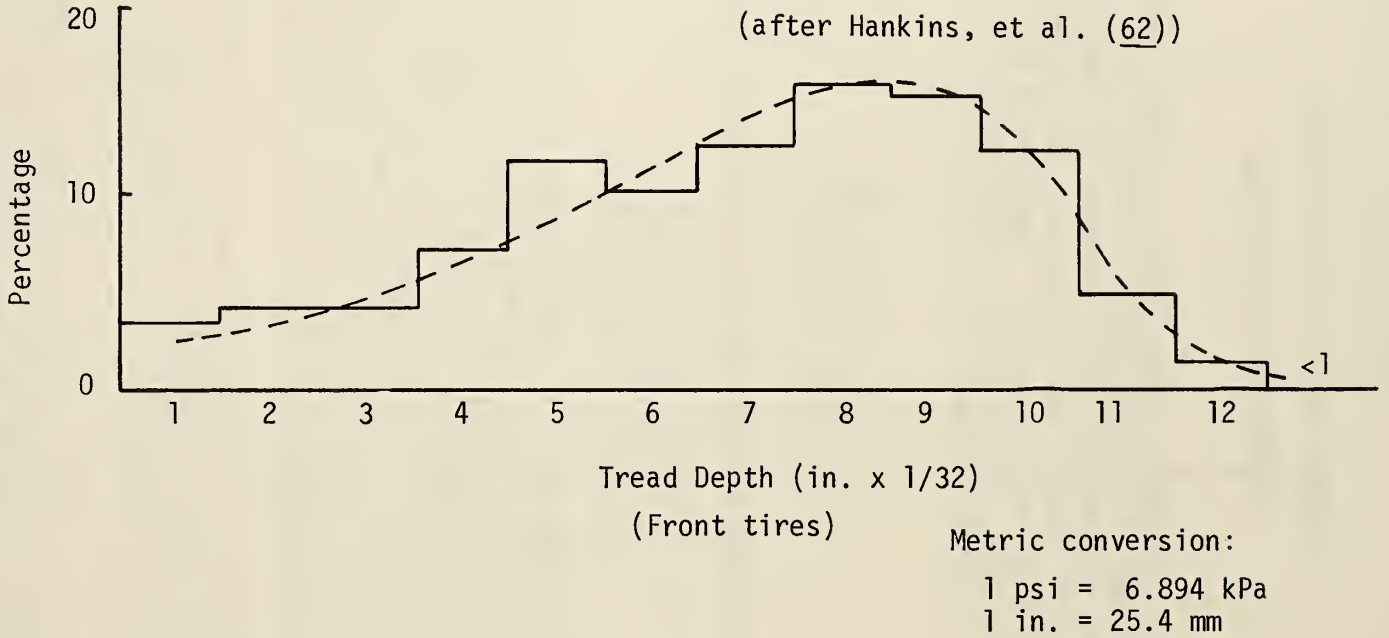


Figure 59. Approximate distributions of tire tread, tire pressure and surface texture for one state.
(continued next page)

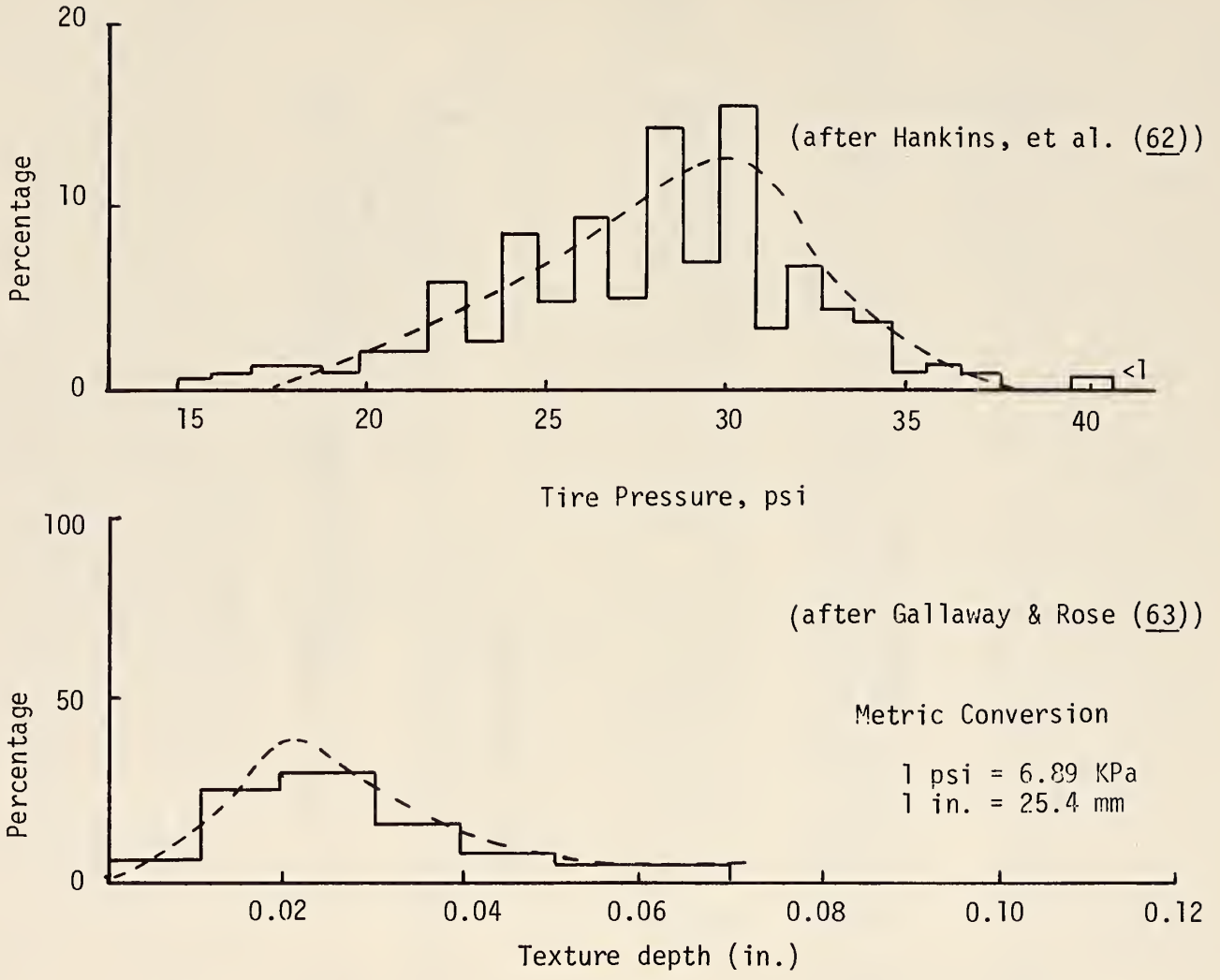


Figure 59. Approximate distributions of tire tread, tire pressure and surface texture for one state.

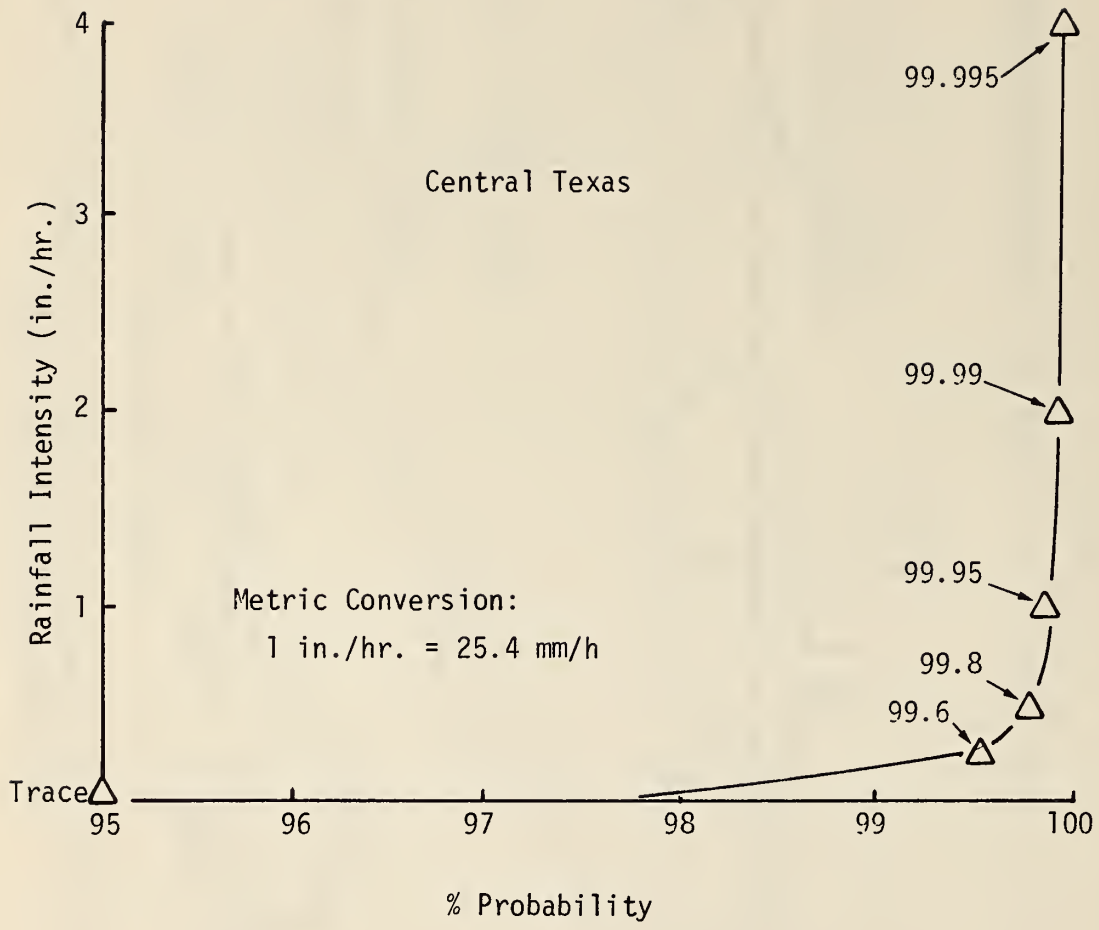


Figure 60. Probability of rainfall in Central Texas.

Summarizing these data, Table 17 shows the probability of rainfalls of different intensities varying from a trace to 4 in./hr. (101.6 mm/h), and also the percentile of each intensity with respect to all rainfall. Interpreting the column labeled "Percentile of All Events Less Severe", 99.9 for 3/4 in./hr. (19 mm/h) means that 99.9% of the time, rainfall intensities less than 3/4 in./hr. would be expected. (Note this includes times when there is no rain at all.) Under the column "Percentile of All Rainfalls Less Severe", 98 for 3/4 in./hr. (19 mm/h) means that 98% of the time when it is raining the rainfall intensity would be less than 3/4 in./hr. (19 mm/h).

Information of this sort for all states could allow highway engineers to select a specific "design rainfall" based on the probability of its occurrence which would be used in geometric designs and surface designs precluding the development of significant positive water depths on the roadway.

Fortunately, information is available which will allow the relationship of different rainfall intensities to depth of water on the pavement as a function of surface slope, runoff length and texture. The appropriate equation which was developed by Gallaway (11) is:

$$WD = .00338 [TXD^{0.11} L^{0.43} I^{0.59} S^{-0.42}] - TXD.$$

Where

WD = the water depth above the top of the surface asperities in inches (mm/25.4)
L = runoff length in feet (m/.305)
TXD = texture depth in inches (mm/25.4)
I = rainfall intensity in in./hr. ((mm/25.4)/h)
S = slope of surface in ft./ft. (%)

Using this equation, Figures 61 through 66 were developed which give those combinations of road cross slope and texture which would result in a zero water depth for the specified rainfall intensities. For example, Figure 63 is prepared for rainfall intensities of 3/4 and 1 in./hr. (19 and 25.4 mm/h). In the set of curves for 1 in./hr. (25.4 mm/h) the curve labeled 36 gives those combinations of cross slope (ordinate) and texture (abscissa) necessary to maintain zero water depth with a runoff length of 36 feet (11 m). Note for a cross slope of 1/8 in./ft. (1%) a texture of 0.082 in. (2.08 mm), identical to the average texture depth, would be present on the pavement.

The relationships given by Figure 61 through 66 can ultimately be used as a basis for anti-hydroplaning criteria, but other relationships must be developed for each state before implementation can be achieved. One of the most critical elements is in formulating the basis for determination of a design rainfall intensity and then to make this determination for each state or geogrphic area.

Table 17. Probability of events and percentile rainfall.

I in./hr.	Percentile of All Events Less Severe	Percentile of All Rainfalls Less Severe
Trace	95	0
1/4	99.6	92
1/2	99.8	96
3/4	99.9	98
1.0	99.95	99
1.5	99.97	99.4
2.0	99.99	99.8
4.0	99.995	99.9

Metric Conversion: 1 in./hr. = 25.4 mm/h

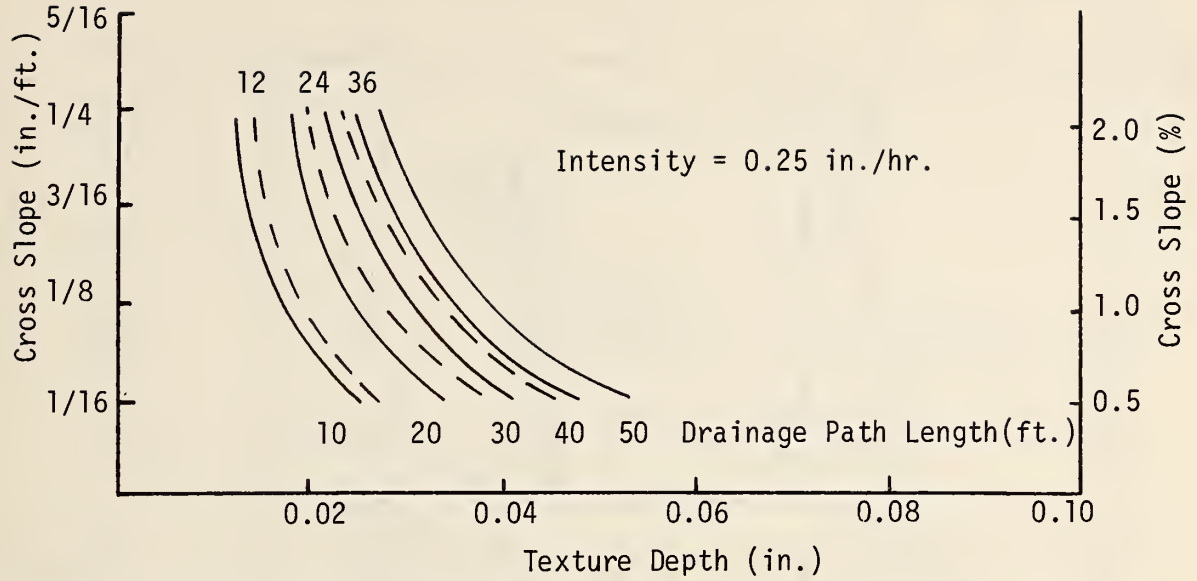


Figure 61. Curves of zero water depth for an intensity of 0.25 in./hr.

Metric conversion:

1 in./hr. = 25.4 mm/h

1 ft. = 0.305 m

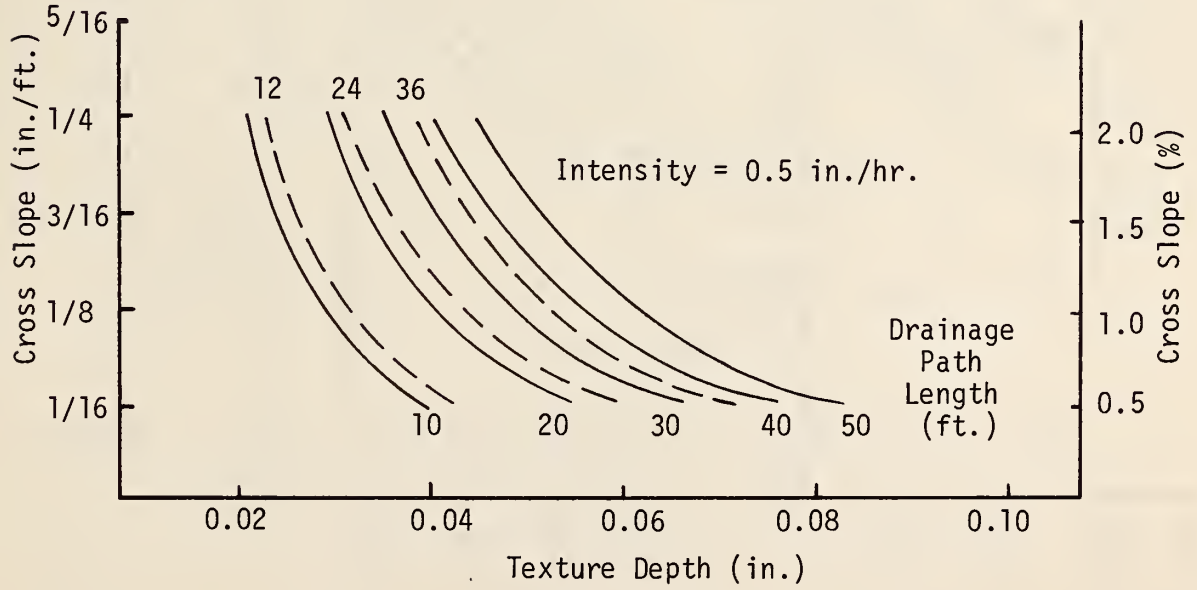


Figure 62. Curves of zero water depth for an intensity of 0.5 in./hr.

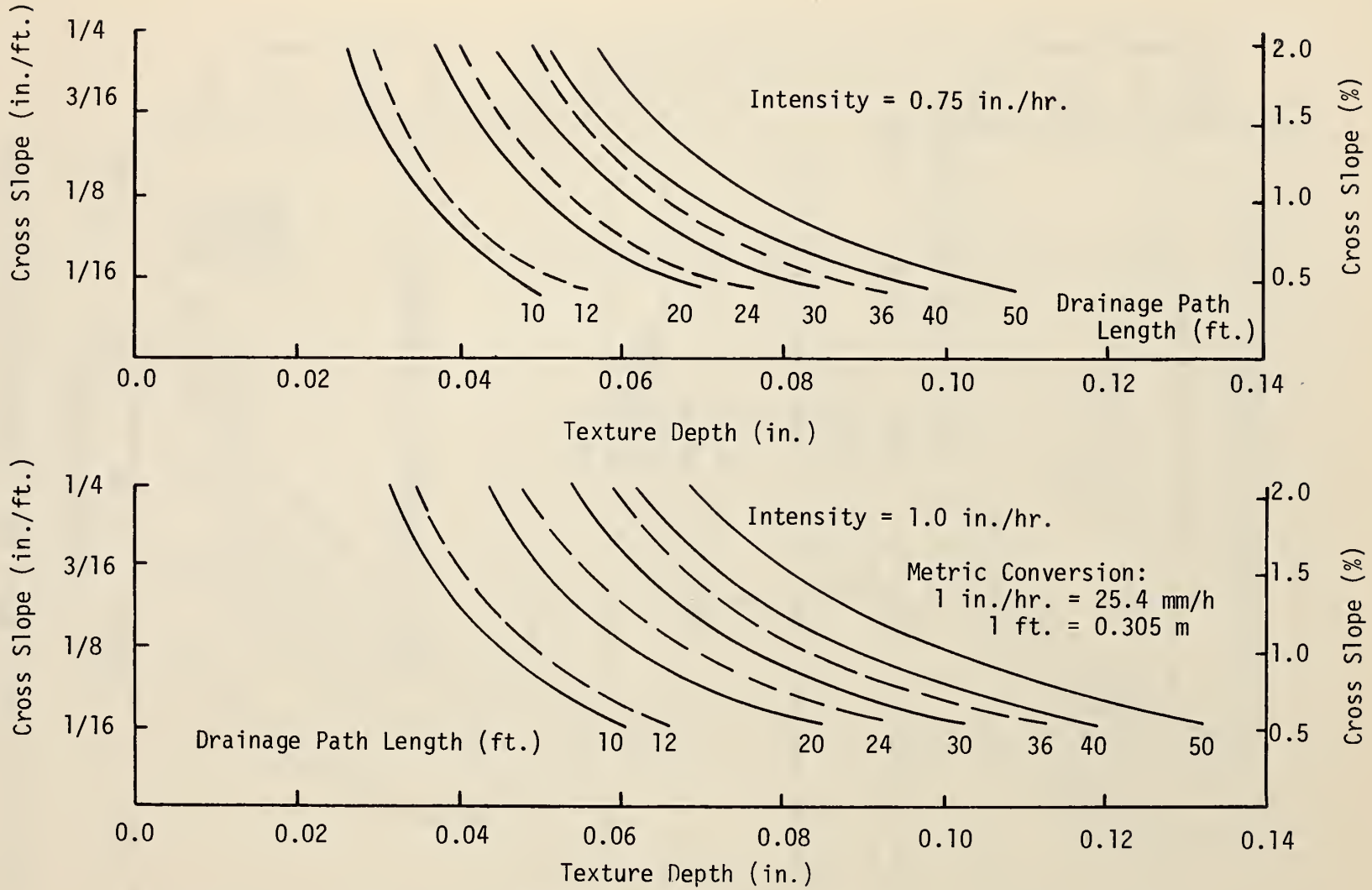


Figure 63. Curves of zero water depth for intensities of 0.75 and 1.0 in./hr.

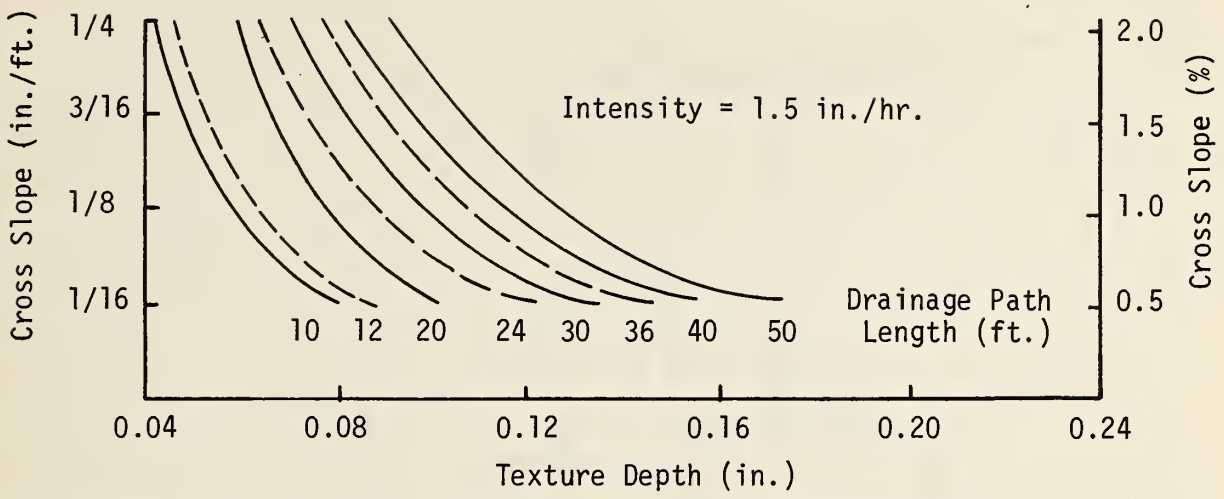


Figure 64. Curves of zero water depth for an intensity of 1.5 in./hr.

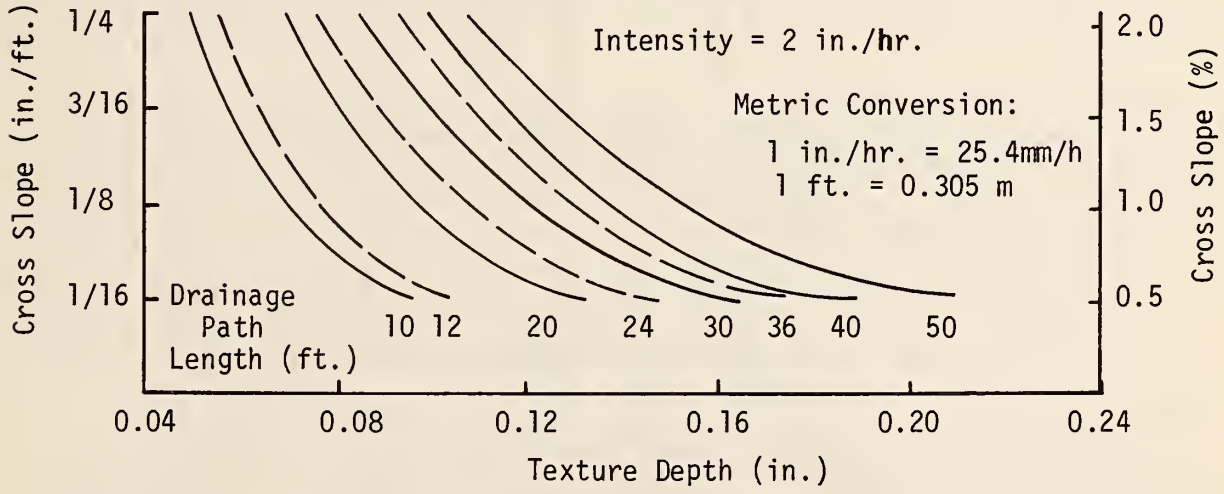


Figure 65. Curves of zero water depth for an intensity of 2 in./hr.

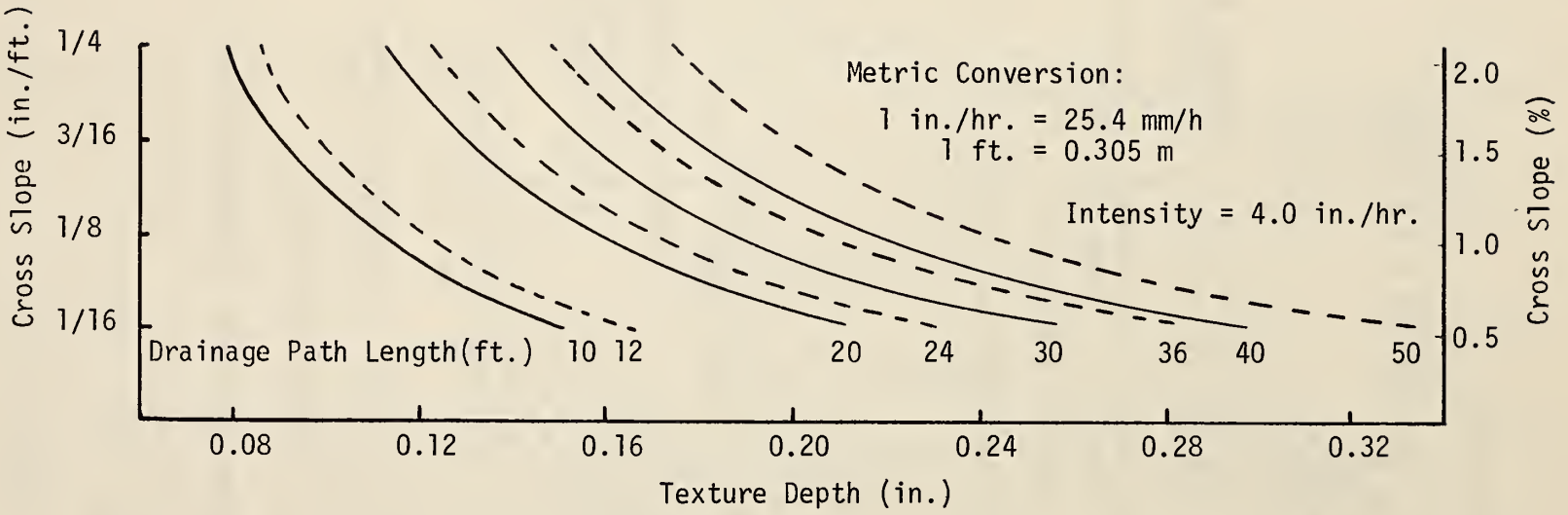


Figure 66. Curves of zero water depth for an intensity of 4.0 in./hr.

As an example of how such a criteria might be applied using rainfall data from Central Texas, consider the following: Figure 60 shows the relationship between rainfall intensity and the probability of the event.

On the upper end of the scale, rainfall intensities of over 1 in./hr. (25.4 mm/h) are such rare occurrences that there would seem to be a rather remote chance that design for these events could be justified from an economic standpoint, although in other areas of the country frequency of this intensity may be high enough to justify accommodation of the condition. In the current example, however, a 1 in./hr.(25.4mm/h) intensity is a .05% probable event and is the 99 percentile rainfall (99 out of 100 rainfall events are less severe). Therefore it seems reasonable that the range of choice for a design rainfall could be reduced to between 1/4 and 1 in./hr. (6.4 and 25.4mm/h). A reasonable compromise might be an intensity of 1/2 in./hr. (12.7 mm/h). This corresponds to a 0.2% probable event and is the 96th percentile rainfall. Figure 62 shows that cross slopes of 1/4 in./ft. (2%) and drainage path lengths up to 36 ft. (11 m) could be accommodated if texture were maintained over .040 in. (1.02 mm) For the lower cross slope value of 1/16 in./ft. (0.5%) texture of .070 in. (1.79 mm) needs to be maintained, thus emphasizing the value of the larger cross slopes since only open-graded mixes and seal coats are likely to exhibit textures of .070 in. (1.79 mm) or above. Note that these considerations assume a planar road surface rather than one which is puddled or rutted. Certainly this is a rather optimistic assumption, showing the need for consideration of the frequency of these factors before a comprehensive criteria can be developed. The construction and maintenance of texture is one of the most serious problems. Figure 59 indicated that a mean value of texture on a specific group of roads (62) is only about .025 in.(0.64 mm).

Based on the relatively simple analysis presented, it seems likely that significant additional resources are not required for highway engineers to achieve geometric and surface conditions on new construction that will make hydroplaning an improbable event. However, achieving this goal on extant roadways has critical economic connotations, and is likely to be highly questionable from a cost/benefit viewpoint. Even so, a certain portion of these highways require resurfacing each year due to other shortcomings. When this occurs there seems little reason that an appropriate antihydroplaning criteria should not be met.

Although it is obvious that new construction and resurfacing should be in conformance with criteria that would preclude the occurrence of hydroplaning in all but the most improbable climatological conditions, one other option could be exercised by highway engineers in the interim between discovery of potentially unsafe conditions and the availability of funds necessary to correct the condition. That option is mandatory vehicle speed reductions using warning signs and/or speed zones. This option has been discussed at length by Weaver, et al., (65). If this interim measure is to be used, the question must be answered: What speed is safe for this roadway? The answer to this question must be found by estimating the available friction, a function of speed, for a

specific roadway under a "design" rainfall condition. It must also make use of readily determined data. In approaching this determination of available friction, or available control forces, two zones will be considered: The first will be concerned with speeds below 40 mph (64 km/h), the speed at which penetration of the water wedge can begin, but will not reduce friction significantly; and speeds above 40 mph (64 km/h) where water wedge penetration is significantly reducing available friction up to the speed where a full loss occurs. Under HPR 163, Hayes has predicted lower boundaries of vehicle accelerations (equal numerically to average available friction) for vehicles representative of the population, for absolute* water depths of approximately .1 in. (2.5 mm). These lower boundaries shown in Figure 67 show that SN_{40} is a conservative estimate of available friction for speeds less than 40 mph (64 km/h) and for SN values less than 40 (the most critical range of pavement SN values). This conclusion is reinforced by several other studies (65,66).

At speeds greater than 40 mph (64 km/h) available friction can decrease rapidly as full hydroplaning is approached. However, this decrease is not as rapid as the slope of a straight line projected from SN_{40} at 40 mph (64 km/h) to zero friction at full hydroplaning speed. Therefore such a line would also be a conservative estimate of available friction from 60 mph (64 km/h) up to hydroplaning. This construction is demonstrated by Figure 68. A slightly more sophisticated approach would be to use the values of acceleration predicted by skid number in Figure 67 to give the available friction boundary in the 0 to 40 mph range (64 km/h).

The construction is made after the following values for a specific site have been either set or determined.

ASTM Skid Number, SN_{40}

Texture Depth, TXD - Putty Impression or Sand Patch

Drainage Path Length, L

Slope of L, S

Design Rainfall Intensity, I**

Design Tread Depth, TD***

Design Tire Pressure, P***

---

*Representative of an average quantity of water over the surfaces and not measured with respect to top of asperities.

**An intensity of 1/2 in./hr. (13 mm/h) was previously suggested, along with a recommendation that each state analyze rainfall records in order to select a design rainfall.

***Since some states now have a minimum legal tread depth of 2/32 in. (2 mm) which corresponds to the lower 2.5% level of the population (Figure 59), this number might be used as the "design tread depth". Further, if a 2.5% population level is used for tire pressure, a design pressure might be set at 20 psi (138 kPa) (Figure 59).

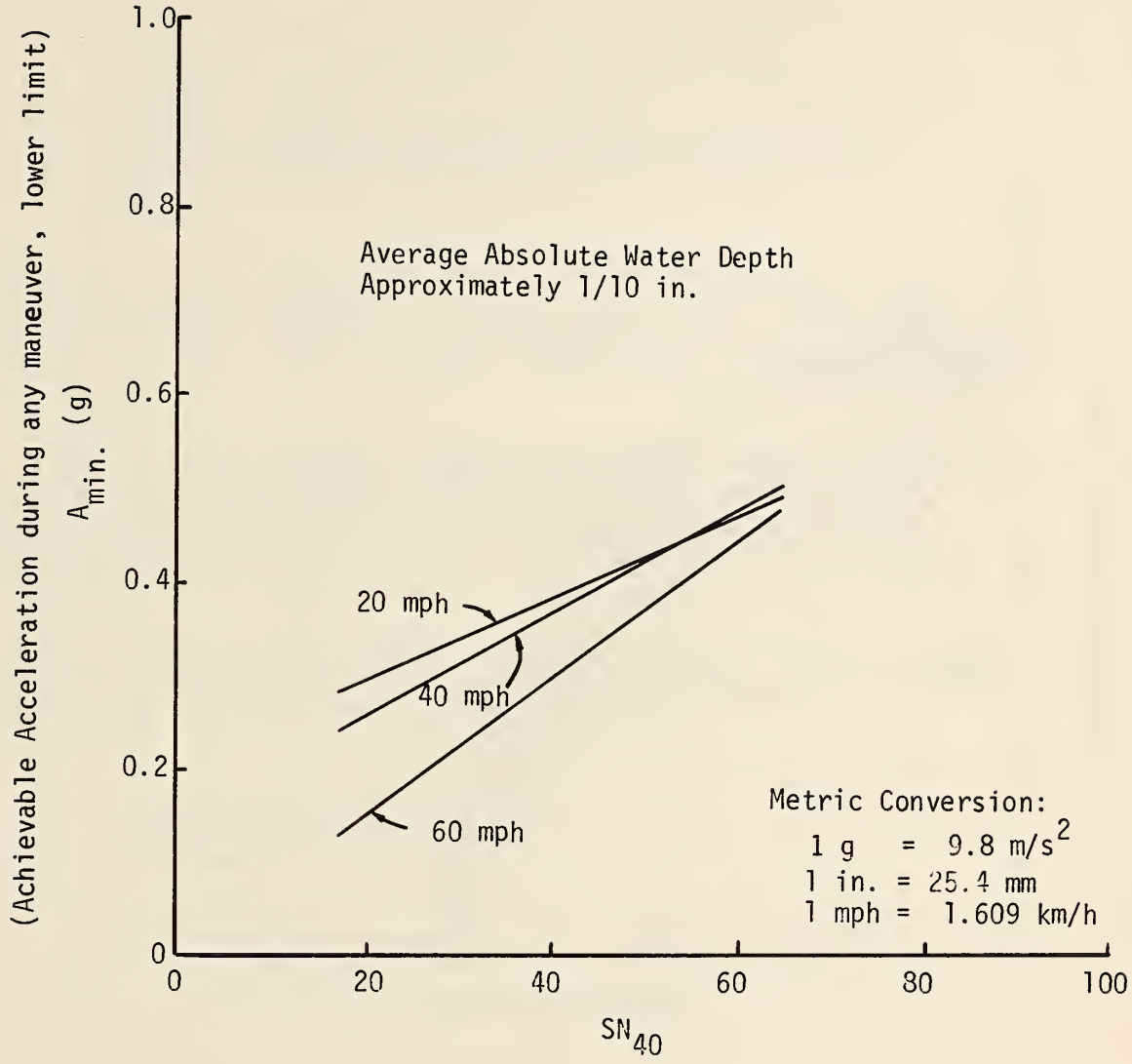
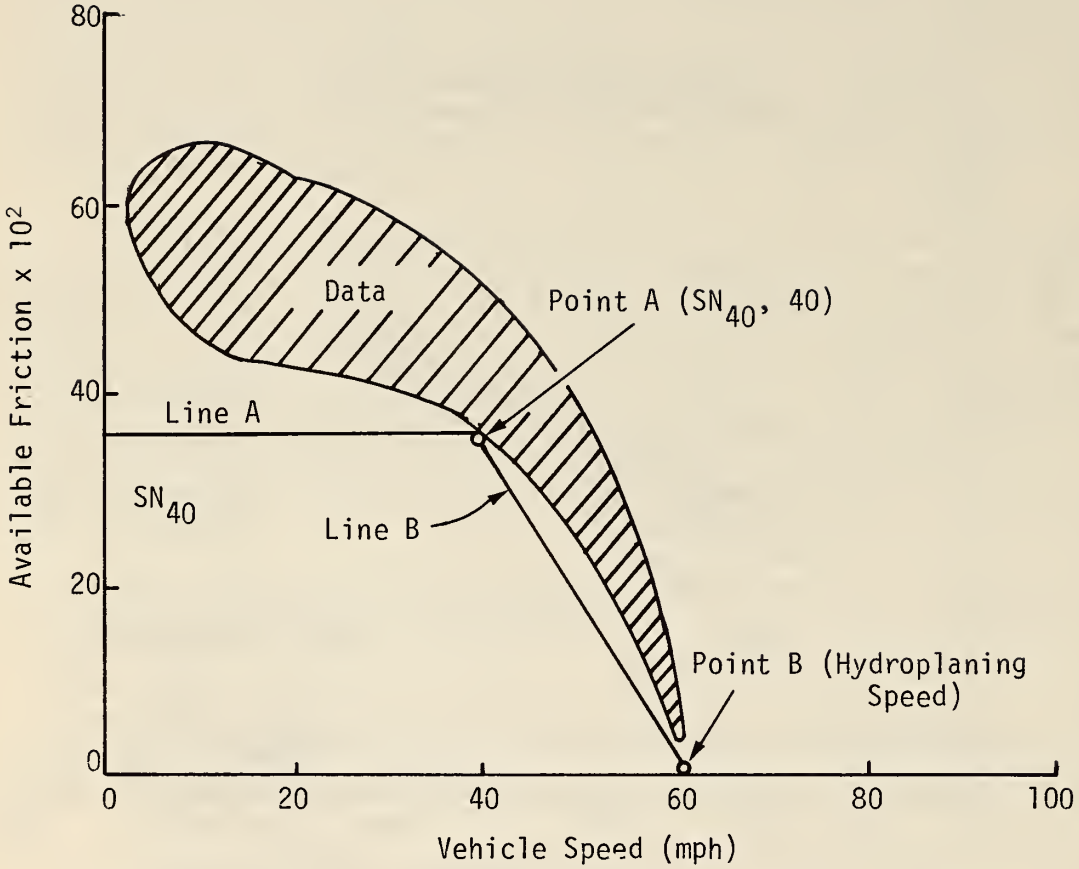


Figure 67. Estimated acceleration boundaries as a function of SN_{40}.



Metric Conversion:
1 mph = 1.609 km/h

Figure 68. Construction of available friction envelope.

Three steps are followed in making this construction (Figure 68):

1. Line A is drawn from the value of SN_{40} on the ordinate horizontally to 40 mph (64 km/h) shown by Point A.*
2. Point B is calculated using equation 18 to determine water depth and equation 3 to determine the velocity for 10% spin down (the designated indicator of hydroplaning).
3. Points A and B are connected by Line B which closes the envelope.

The shaded, "Data", area is indicative of the conservative nature of the available friction envelope. The actual speed limit should then be set based on the requirements for friction of the various maneuvers expected at a specific site. These friction requirements are detailed by Weaver, et al., (65).

Line B, for any specific site, can also be interpreted as the available accelerations, average friction or average control forces in the range of partial hydroplaning for a certain element of the vehicle population, in this case all but the 2.5 percentile as indicated by the data developed by Hankins (62).

The concepts of available friction and required friction used in the format suggested will then allow an analysis of any specific site to determine whether corrective action is required.

*The construction is valid only if the value of SN_{40} is equal to or less than 40. (It is shown in the example as 37.) If the SN is greater than 40, line A will not lie below all the data and therefore will not be conservative. In this case the boundary indicated by Figure 67 must be used.

CHAPTER VIII

TENTATIVE RECOMMENDATIONS FOR CONSTRUCTION

In considering the critical dimensions leading to hydroplaning it is obvious that each factor does not operate independently but is part of a complex interaction. Thus critical dimensions for each factor cannot be determined without qualifying assumptions relating to the other factors. In attempting to arrive at critical dimensions relating to the need for changes in construction practice and/or traffic control, certain factors will be set as tentative recommended design conditions. In general these will be the factors outside the control of highway engineers. The following design conditions are recommended:

1. Vehicle Speed - the speed limit. In most cases this speed will not be greater than 55 mph (88 km/h) for rural highways.
2. Vehicle Path - reasonably smooth. (There seems little hope of accommodating highly erratic maneuvers during periods of rainfall.)
3. Tire Tread Condition - 2/32 in. (1.6 mm). This is the minimum legal tread depth in a number of states.
4. Tire Pressure - 20 psi (137.9 kPa). According to preliminary data, 97.5% of the vehicles would have higher tire pressure.
5. Tire Type - conventional bias ply and radial designs. Tires developed for specialized uses such as racing may prove to be too demanding of pavement drainage.
6. Rainfall Intensity and Frequency - the 96 percentile rainfall. The value of rainfall intensity corresponding to this level would be a geographic variable. In Central Texas the 96 percentile corresponds to an intensity of 1/2 in./hr. (12.7 mm/h) which would be encountered 0.4% of the time. (See Table 17, Chapter VII.)

Vehicle speed does not enter into the antihydroplaning criteria for texture and cross slope since these criteria are concerned only with maintaining water depths that will not cover the pavement asperities. Speed is of critical importance to the analysis of specific sites to determine the potential for hydroplaning, in which case the computed hydroplaning speed may be compared to the posted speed limit. This proposal assumes that if the hydroplaning speed is greater than the posted speed limit the obligation of the highway engineer is fulfilled; if it is less than the posted speed limit, corrective action is required. A higher design speed could be set at the engineer's discretion following the AASHO precedent for facility design speed significantly above the posted speed. However, considering the infrequency of the recommended design rainfall, it is not believed that higher design speeds are justifiable.

Based on the information which has been presented in the foregoing chapters and the stated design conditions, recommendations can now be

made for combinations of pavement slope, drainage path length and texture. For impermeable surfaces, recommendations can best be summarized by Figure 62, Chapter VII, which gives relatively complete formation of positive water depths. However, such a figure is of more value in the analysis of a situation rather than for use as a positive design criterion. Therefore, a summary is given by Table 18 with discussion of the recommendations in the following paragraphs.

Dense Surfaces (Impermeable)

The recommended minimum tentative surface macrotexture is 0.040 in. (1.02 mm) as determined by the modified sand patch or silicone putty method.

The recommended tentative minimum cross slope for two-lane and four-lane rural facilities is 1/4 in. per ft. (2%). For multilane rural highways of six or more lanes, it is recommended that the cross slope for the inside lane be reduced to not less than 1/8 in. per ft. (1%) and that the outside lane not exceed 5/16 in. per ft. (2.6%). All other lanes should be 1/4 in. per ft. (2%).

Acceptable noise levels would probably be the limiting factor for maximum macrotexture values. The acceptable noise level in urban areas would be generally lower than that for rural areas. Since different types of tires generate different levels of noise on the same pavement and since tire types are constantly changing, it would be difficult to select a maximum macrotexture value. Another input to tire noise generation is the type of surface independent of the magnitude of the texture value. For example, chip seals generate higher noise levels than open-graded mixtures of the same measured macrotexture.

In rural areas, a suggested tentative maximum macrotexture for the noisiest type of surface (chip seal) is 0.150 in. (3.8 mm). For urban areas this value should be reduced to 0.100 in. (2.5 mm).

Open Surfaces (Permeable)

As a general rule, pavement surfaces which consist of open-graded mixtures will have macrotexture in the range of 0.040 to 0.120 in. (1.02 to 3.0 mm); however, certain fine-graded mixtures have been designed to perform as permeable surfaces and such surfaces would have very limited macrotexture, possibly less than 0.01 in. (0.25 mm).

Based on a theoretical analysis, the rainfall intensity required to flood the "average" open-graded surface course with a 36 ft. (11 m) drainage path is approximately 0.01 in./hr. (0.25 mm/h). Yet, performance appears to be impaired only slightly by limited flooding of the surface.

Table 18. Minimum texture and cross slope recommendations.
(continued on next page)

Dense Surfaces			
Facility	Minimum Macrotexture	Cross Slope	Microtexture
Two Lane	0.040 in.	1/4 in./ft. (2.0%)	THD Method
Four Lane	0.040 in.	1/4 in./ft. (2.0%)	Values range from 28 to 47 as vehicle passes per lane year ranges from 5×10^4 (See Table 19).
Multi Lane (Six or more)	0.040 in.	Inside Lane Not less than 1/8 in./ft. (1.0%) Outside Lane Not more than 5/16 in./ft. (2.6%) Other Lanes 1/4 in./ft. (2.0%)	

Table 18. Minimum texture and cross slope recommendations.

Open Surfaces			
Facility	Minimum Macrotexture	Cross Slope	Microtexture
Two Lane	0.050 in.	Not less than 1/8 in./ft. (1.0%)	Same as for Dense Surfaces
Four Lane	0.050 in.	Not less than 1/8 in./ft. (1.0%)	
Multi Lane	0.050 in.	Same as for Dense Surfaces	

It is therefore recommended that minimum tentative surface macrotexture for open-graded surfaces be set at 0.050 in. (1.3 mm)*.

Cross slope requirements of surfaces with open-graded mixtures should be generally less demanding than those for dense surfaces.

Requirements for rural two-lane and four-lane facilities wherein all lanes are surfaced with open-graded mixtures a tentative minimum cross slope of 1/8 in. per ft. (1%) is recommended. For similar surfaces on rural highways of six or more lanes cross slope recommendations are the same as for dense surfaces of six or more lanes.

Generally, noise is not a problem on open-graded surfaces due to the absence of "percussion cups"; consequently no maximum macrotexture value is suggested for this type of surface. Other constraints on the top sized aggregate used in such mixtures will normally preclude any problem from this source.

Superimposed on these texture and cross slope requirements should be an assurance of adequate microtexture. In the case of portland cement concrete pavements, suggested macrotexture requirements have been made, and in this connection it has been assumed that high quality mortar consisting of 50% or more of acid insoluble silica sand in the No. 30 to No. 100 mesh material assures adequate microtexture. Such is not the case, generally, for bituminous surfaces. Adequate microtexture must come primarily from the coarse aggregate in the mixture. Consequently, a microtexture requirement is needed to assure adequate performance. Control of this variable may be effected by requiring a minimum polished stone value as determined by a standardized procedure. The current method used by the Texas Highway Department (a modification of the British procedure) is recommended.

To be most effective from the viewpoints of cost and performance, more than one polish value should be permitted. Indeed, for low traffic volume facilities a polish value should not be included in the surface design criteria.

Currently minimum polish values specified by agencies using this control are in the range of 35 to 45. Values commonly specified in Texas are 35 to 40, although Texas' experience with some aggregates that meet the minimum value of 35 have not been entirely satisfactory.

The polish susceptibility of a given aggregate as measured by the above recommended procedure is a function of its physical and mineralogical properties. The rate at which this same aggregate polishes on the road is a function of the type and volume of traffic and the effects of the environment. For some aggregates the environmental effects may be quite pronounced while other aggregates may be virtually unaffected.

---

*As measured by the silicone putty method.

A tentative minimum polish value of 37 is suggested for two-lane heavy* traffic with 10 to 15% trucks. A minimum value of 42 is suggested for multilane facilities with heavy traffic, again assuming 10 to 15% trucks.

Medium traffic would require a lower polish value, whereas light traffic would probably not involve a polish value requirement. Suggested tentative polish values and associated traffic volumes are summarized in the following table.

Table 19. Tentative recommended polished stone values for various traffic volumes*.

Traffic, Vehicle Passages Per Lane Per Year	Median Polished Stone Value	Range
50,000 to 150,000	28	26.5 - 29.5
150,000 to 600,000	32	30.5 - 33.5
600,000 to 2,500,000	37	35 - 39
2,500,000 to 10,000,000	42	40 - 44
10,000,000 +	47	47 - 49+

*Accelerated Polish Test for Coarse Aggregate TEX 438 (a combination of British Standard 812 and ASTM E 303, The British Wheel and the British Pendulum Tester).

The range of polish values suggests a permissible variation of plus or minus 5% in this value. Aggregates which are classed to meet the polish values falling between these categories would be relegated to the next lower group.

The traffic volumes and associated polish values are specified and possibly idealized in that it is anticipated that environmental effects

---

*Heavy traffic is defined as 700 to 800 vehicles per lane per hour for a two-lane facility and 1500 to 1600 vehicles per lane per hour for multilane facilities.

will be greater on pavements with lower traffic. Therefore, aggregates of lower polish value subject to lower traffic would be expected to be conservatively rated by the standardized test which does not include the effect of the environment.

Normally, the recommendations of this table would be interpreted differently. That is, any aggregate selected for a specific level of service would be suitable for that level and all less demanding requirements.

It is thought that the specific selection route has the advantage of furnishing the driving public with a more nearly uniform tire-pavement interaction feedback. At the same time this approach presents problems from the aggregate availability viewpoint -- a problem that could easily make this approach economically unacceptable.

Although it is a well known fact, the reader is reminded that the initial surface macrotexture of a pavement changes rather rapidly in the early months of service. The high initial level of surface macrotexture found on portland cement concrete may be expected to be reduced by about 25% in the early months of service, as has been previously stated. Certain plant mixed bituminous pavements exhibit increases in surface macrotexture with service and weather effects, whereas chip seals and open graded plant mixtures may be expected to lose macrotexture with service and environmental effects.

Microtexture of plant mixed bituminous-aggregate mixtures usually increases to a peak value some months after being placed in service, and then decreases to some reasonably constant value dependent upon aggregate characteristics, traffic and the environment. It then becomes evident that the as-constructed surface macrotexture will be generally different from the long term performing texture. Naturally, the recommendations set forth refer to long term texture values.

Construction of Asphaltic Concrete Surfaces

In a recent NCHRP study by Van Til, Carr and Vallerga (67) the authors made recommendations for wear resistant and skid resistant highway pavement surfaces indicating the need to consider the following areas:

1. Skid resistance
2. Hydroplaning potential
3. Effective life
4. Cost
5. Application procedures
6. Applicability of available materials and equipment
7. Maintainability
8. Riding quality
9. Tire noise
10. Tire wear
11. Appearance

The subject study is concerned primarily with skid resistance, yet the factors listed above are considered to be generally involved in most streets and highways with the degree of hazard depending largely on traffic factors, road geometrics and environmental conditions.

Van Til, et al., (67) have recommended a total of ten skid resistant systems of construction for immediate implementation which offer, in their opinion, the greatest promise for success. Of these systems the following are recommended as systems which are at the same time skid resistant and serve to reduce the probability of hydroplaning. Since portland cement concrete pavements are treated separately in this report, variations of such systems will not be considered here.

1. "Open-graded asphalt concrete--a hot plant mixture of asphalt cement and open-graded aggregate, in which special consideration to wear and skid resistance is given in the selection of the aggregate.
2. "Dense-graded asphalt concrete, optimum mix design--a conventional dense-graded, hot plant mixed asphalt concrete in which special consideration to wear and skid resistance is given in the selection of the aggregates and in the design of the mixture.
3. "Skid-graded asphalt concrete--a hot plant mixture of asphalt cement and a skid-graded aggregate, in which special consideration to wear and skid resistance is given in the selection of the coarse aggregate gradation.
4. "Asphalt concrete with rolled-in precoated chips--a skip-graded asphalt concrete mixture, followed by the application after initial placement and before rolling, of a cold aggregate which has been precoated with an asphalt binder, and in which special consideration to wear and skid resistance is given in the design of the asphalt concrete mixture and in the selection of the aggregate to be used as precoated chips.
5. "Asphalt seal coat--the application of asphalt binder followed immediately by an application of aggregate which has been selected with special consideration to wear and skid resistance."

The authors quoted above give details for implementing these systems and describe guides for the preparation of specifications for materials construction procedures which will not be repeated here; however, special considerations relating to the merits of these systems to minimize hydroplaning are in order and will be discussed.

Based on field performance data, open-graded asphalt concrete mixtures appear to offer the greatest promise as a device to minimize hydroplaning.

A mixture design method has been recommended by Smith, Rice and Spelman (68) and this method has been amplified and modified by Gallaway and Epps (69). These researchers stress aggregate selection, gradation and mixture temperature control during construction to optimize the desirable properties of this system. The recommended mixture design procedure minimizes the probability of loss of adequate voids during service; however, there is no assurance that the permeable voids in such mats will not be clogged by infiltrated debris. Should the voids become clogged the antihydroplaning properties will be reduced to that of an equivalent dense-graded mixture with comparable macro-microtexture properties. For this reason it is quite important that provisions in the aggregate selection consider this probability.

Open-graded mixtures are considered suitable for heavy traffic both urban and rural, and the service life would normally be expected to fall in the five to ten year range. This type mixture is usually placed as a thin overlay on an existing surface to improve friction and minimize hydroplaning but at a unit cost much above that of conventional asphalt concrete. Because total water drainage from such surfaces is delayed, it is critically important that the surface being overlaid be waterproof, otherwise structural distress will result. As is the case with all thin mats, field compaction may be a problem.

Dense-graded asphalt concrete is widely and successfully used as a pavement surfacing, and its skid performance is well documented. Primary reliance is placed on the coarse aggregate for adequate friction. Where dense mixtures are used, emphasis should be placed on the top size of the aggregate grading. Practices by many agencies during the past decade indicate that the top size of the aggregate grading is in the 3/8 to 1/2 in. size (10 to 13 mm). For better performance under inclement weather conditions a top size of 5/8 in. (16 mm) is recommended. It is further recommended that the mortar fraction of this type mixture be composed of aggregates that wear at distinctly different rates. Couple this active mechanism with a nonpolished large coarse aggregate and the result will be an optimization of this particular system.

Skip-graded asphalt concrete surfacing functions most effectively in those designs where the coarse aggregate predominates -- assuming that this fraction of the mixture is a nonpolishing and durable material. Because such designs are often somewhat open (high void content) care must be exercised to avoid their use on pavement structures susceptible to water damage. Again the top size of the coarse aggregate is important. Larger top size will assure better drainage with tire noise a primary constraint on the maximum size. Mixtures of this type would normally contain between 60 and 70% plus No. 8 mesh material with a somewhat above average amount of high viscosity binder. Production and placing of this type mixture should present no special problems.

Precoated chips rolled into asphalt concrete offer an inexpensive approach to improving both friction and drainage in those areas where nonpolishing aggregate is scarce or expensive. Extensive use of this

concept has been practiced in the British Isles (70), and it has been introduced in the United States within recent years on a limited scale (71, 72). Conventional asphalt concrete with marginal friction properties may be upgraded in both friction and drainage by inclusion of the precoated chip sprinkle treatment at the time of construction. Recent experiments (71) reveal effective improvement in friction with a very nominal coverage of the high friction chips. The precoated chips were sized 1/2 in. (13 mm) to No. 4 and were spread at the rate of about 25% of the amount required to cover the surface one stone deep. After about 900,000 vehicle passages the treated surface had SN_{40} values about 50% above the control section.

Possible disadvantages of this system include excessive embedment of the precoated chips, possible inadequate bonding of the chips to the substrate and excessive wear of the chips. To maximize the drainage potential of this system it would be necessary to use large chips, the embedment of which would be critical. A hot mat that would accept large chips would probably have to be gap graded or fine grained as opposed to a harsh mixture. Water susceptibility could be a problem. This system would be ranked with dense-graded, coarse textured asphalt concrete for its antiskid potential. However, the system involves additional construction equipment and considerable judgment in the materials selection and timing of construction operations to assure uniform quality and good durability of the system.

Seal coats with cover aggregate offer an inexpensive system for improved surface drainage and high friction when properly designed and constructed with quality materials. Such surfaces will take surprisingly heavy traffic in rural areas where grades are not very steep nor curves too sharp. They are not suitable for heavy urban traffic or traffic involving frequent starts, stops, lane changes, sharp curves and steep grades.

For equal cross slopes, chip seals constructed with rounded aggregates will drain much more effectively than an equivalent surface made with crushed material. Maximizing the microtexture of either of these two aggregate types will minimize the flow of water across a given road of fixed and equal cross slope.

Experiments under simulated rain revealed the following relative performance of two chip seals placed on equal cross slopes -- one composed of uniform-graded, rounded river gravel and the other a uniform-graded, crushed lightweight synthetic aggregate (heat expanded shale). The gravel seal had a measured macrotexture of 0.150 in. (3.8 mm) compared to 0.135 (3.4 mm) for the crushed synthetic (high microtexture) aggregate. At equal cross slopes, at a 24-foot (7.3 m) drainage length and under equal rainfall intensity, the water level on the gravel surface was 1.2 mm below the asperity peaks and the water level on the crushed material was 1.4 mm above the asperity peaks; however, SN values measured at 20, 40 and 60 mph (32, 64 and 97 km/h) ranged 10% higher for the inundated crushed aggregate surface with high microtexture.

Observations which followed cessation of rain revealed an advantage for the rounded gravel chip seal. It drained much more quickly and performed essentially as a dry surface in less than one third of the time required for the crushed lightweight material.

Bond tenacity of seal coat cover stone is related to binder viscosity, embedment depth, stone size and surface textural characteristics. High friction cover stone is subject to high in-service shear forces and therefore care must be exercised in design and construction operations to see that bond tenacity is optimized. Bond tenacity may be improved by the addition of rubber to the asphalt cement or by substituting for asphalt such special binders as epoxies. The added expense of modified or special binders is easily justified in demanding situations such as bridge decks, compound curves and in heavy channelized traffic.

Precoating of cover stone is advised to enhance early establishment of bond, and under certain environmental conditions preheating (to about 300°F (150°C)) the stone assures quick embedment and positive attachment.

Construction of Portland Cement Concrete Surfaces

The construction techniques used to achieve the deeper textures on PCC discussed in Chapter IV appear to pose no problems in terms of either cost or technique simplicity. The various test sections constructed using metal tines, plastic brooms or brushes were constructed in either the longitudinal or transverse direction using available equipment (49). Texture depths as great as 0.081 in. (2.1 mm) were constructed on full-scale test sections using the metal tines spaced closer than 1/2 in. (13 mm) apart, either 1/4 in. (6 mm) or 1/8 in. (3 mm) clear spacing. If plastic grooving was employed, present technology requires the use of a special machine (39) which can only be used on metal forms. This would preclude the use of slip form pavers. Excellent surfaces have been reported with the use of the plastic grooving machine (39) but it is doubtful that it will gain widespread acceptance as long as metal tines can be shown to do the job in conjunction with slip form paving.

In terms of the construction procedures, a separate finishing machine should be required which will follow the paver just ahead of the curing machine. To properly impart texturing to the surface at the right times, this machine (such as the CMI TC-140 Autograde Texturing/Curing Finisher) should not be used as the curing machine because the two operations may occasionally be required simultaneously in different places. The additional cost for this separate machine is estimated to be almost insignificant to the total cost of the project, perhaps $0.02/sy of concrete surface.

Roadway wear would be a problem, as discussed briefly in the previous section. Concrete pavements not exposed to studded tires may be expected to wear from 25 to 35%, based on initial texture depths. A tendency to level off is then observed (49). This wear down does not appear to be any

greater for one type of texture than another. Therefore, to offset this problem, the only solution presently available is to build in extra texture to prolong the useful life of the surface.

APPENDIX A

ALIGNING TORQUES FOR TANGENT PATH

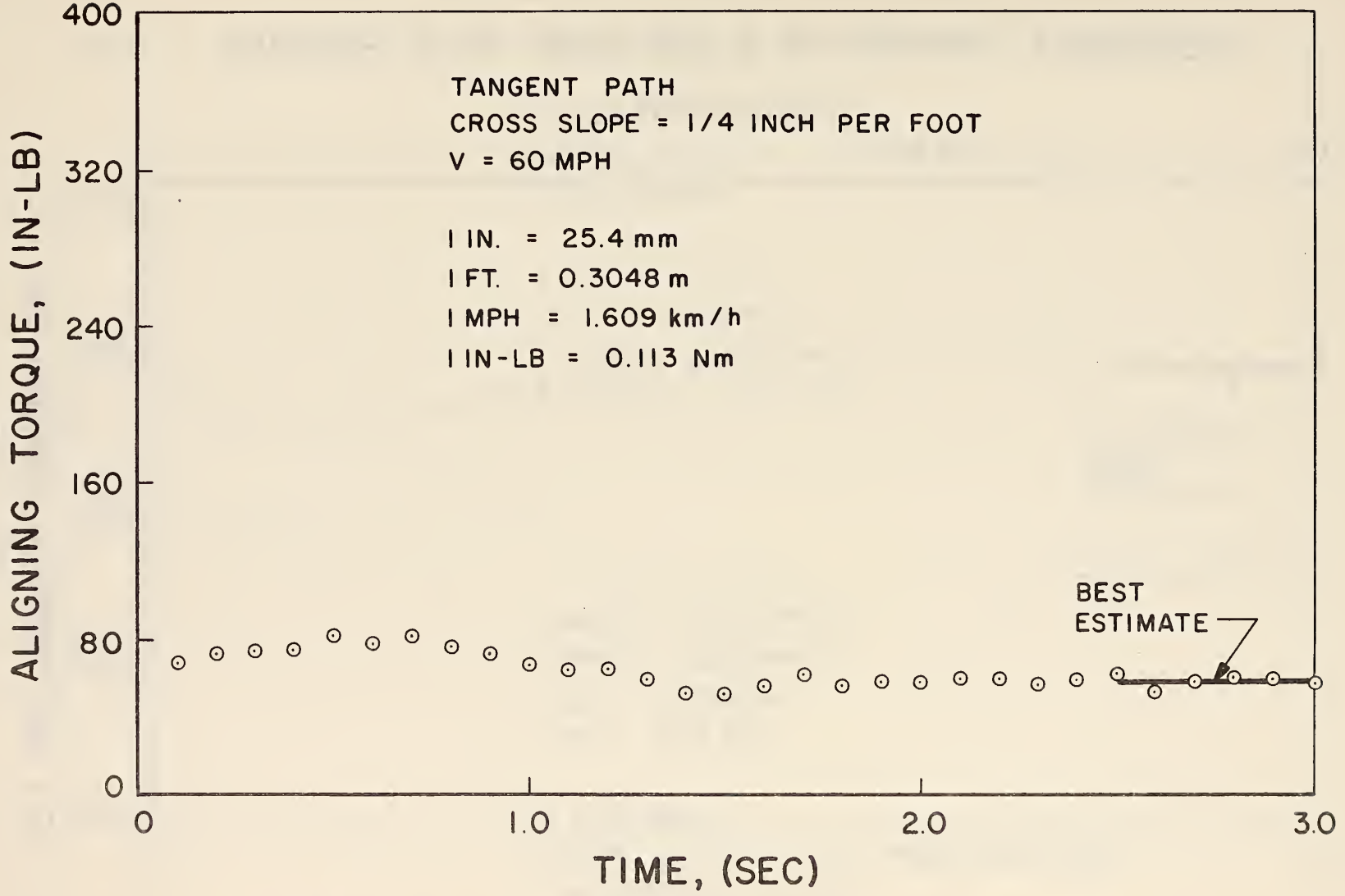


FIGURE A-1. RUN NO. 2 ALIGNING TORQUE

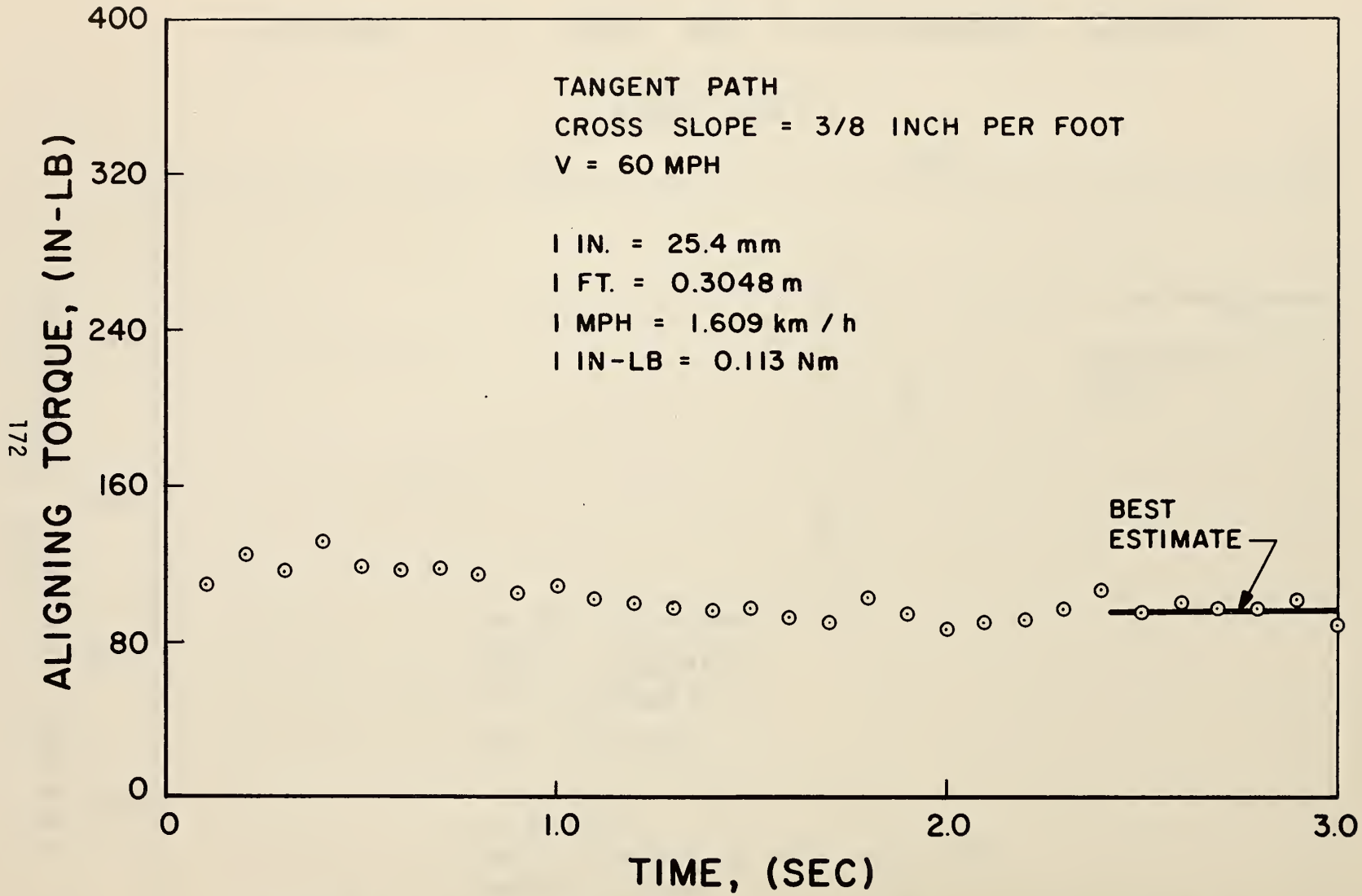


FIGURE A-2. RUN NO. 3 ALIGNING TORQUES

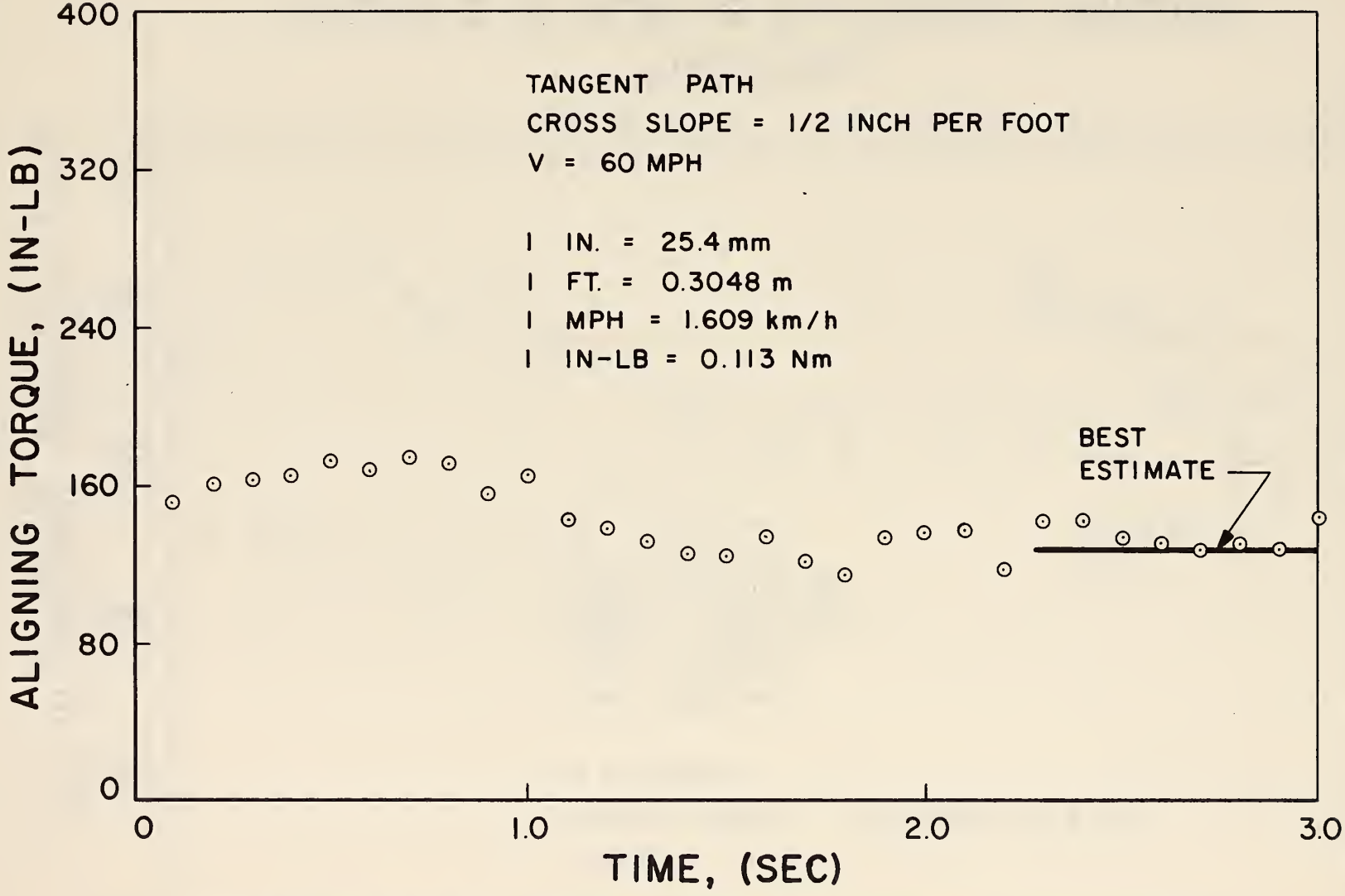


FIGURE A-3. RUN NO. 4 ALIGNING TORQUES

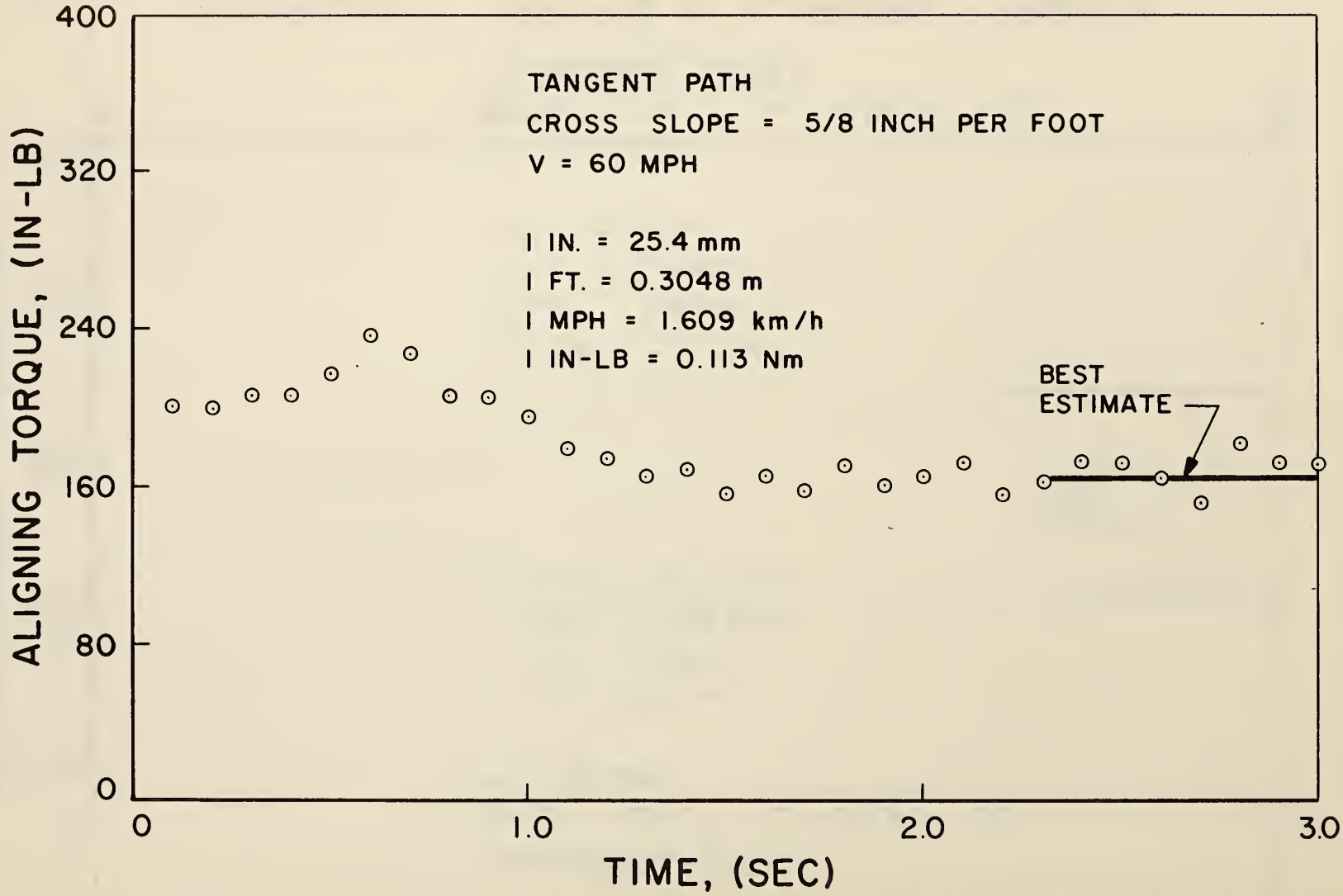


FIGURE A-4. RUN NO. 5 ALIGNING TORQUES

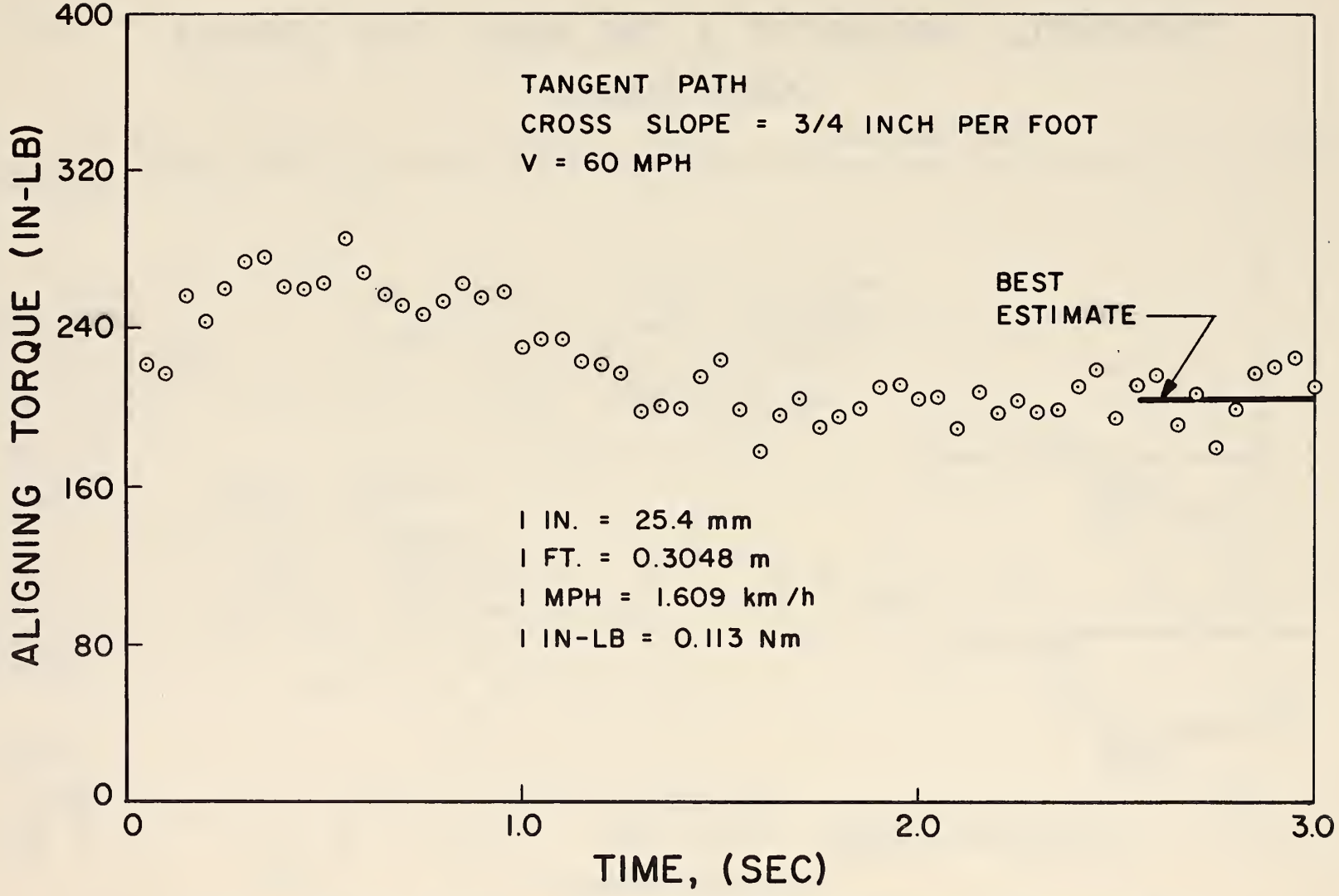


FIGURE A-5. RUN NO. 6 ALIGNING TORQUES

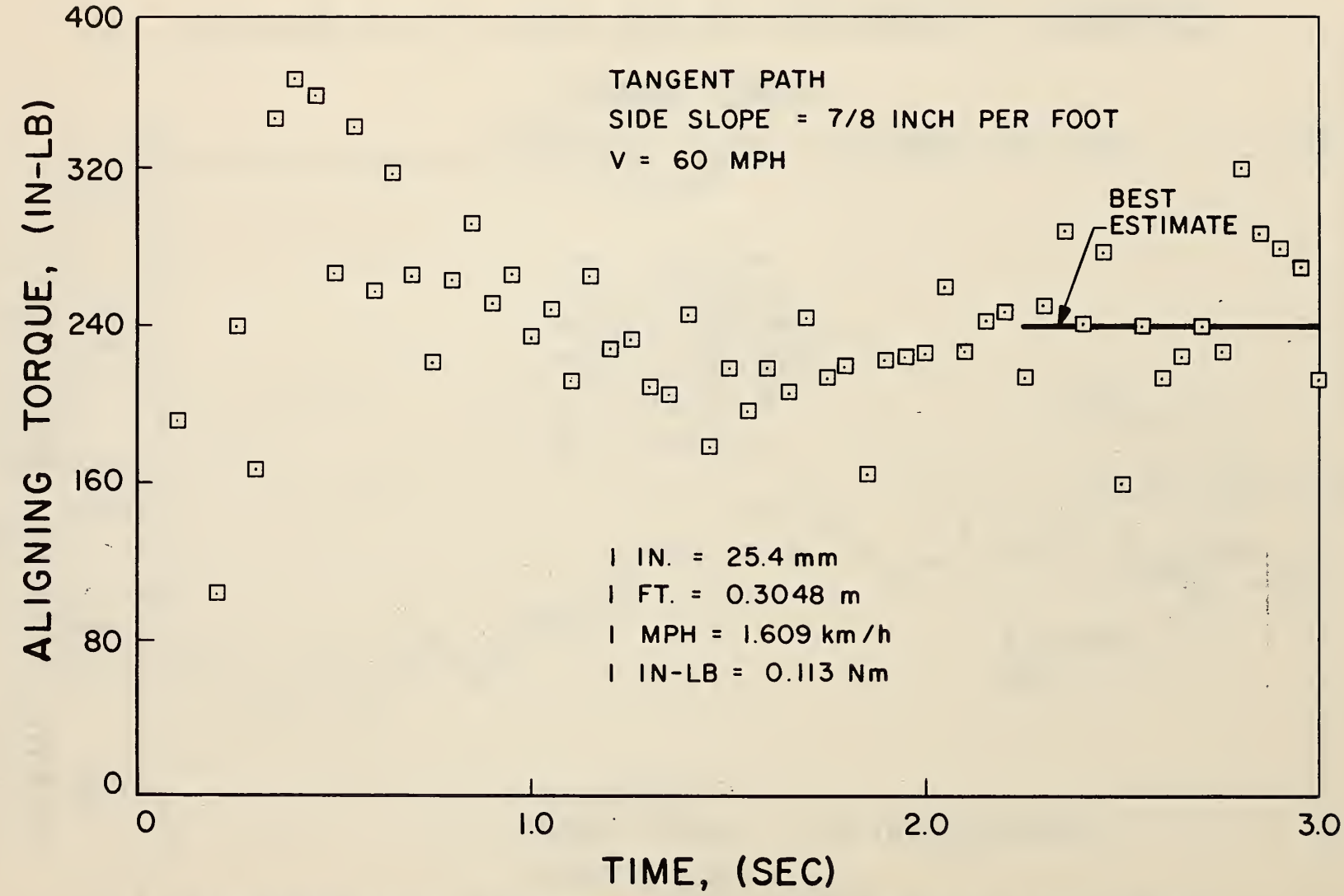


FIGURE A-6. RUN NO. 7 ALIGNING TORQUES

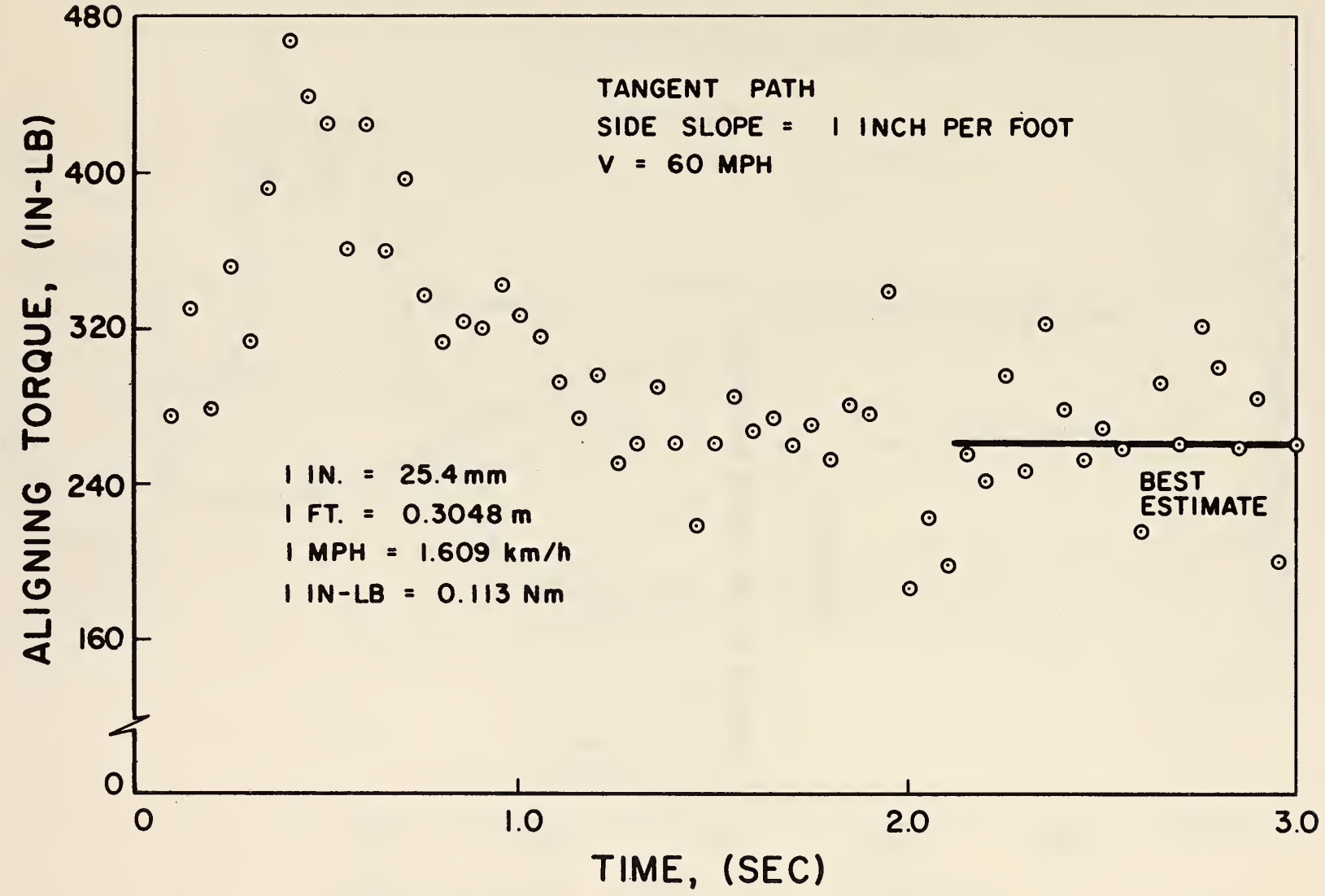


FIGURE A-7. RUN NO. 8 ALIGNING TORQUES

APPENDIX B

RESULTS OF LANE CHANGE MANEUVERS

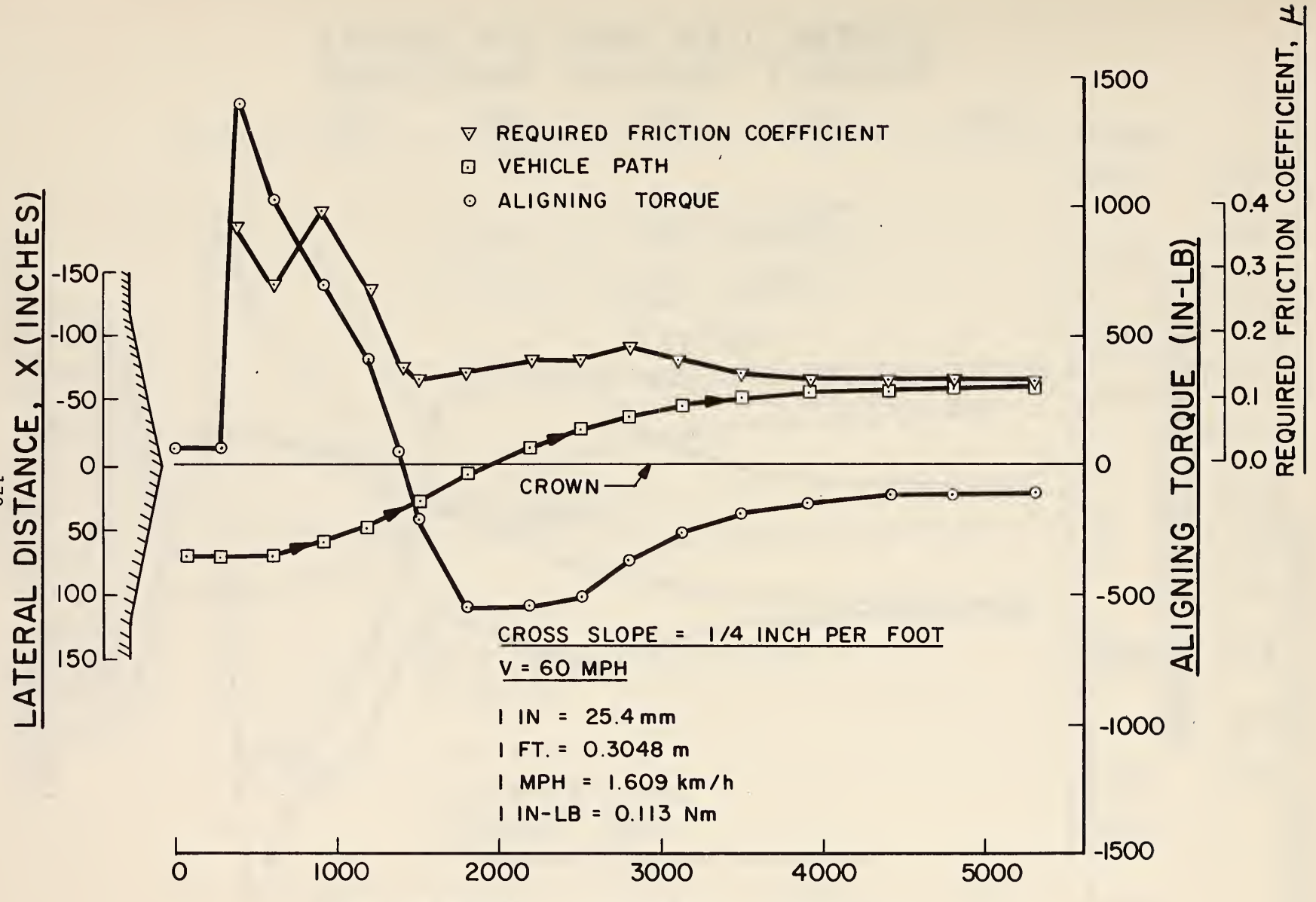


FIGURE B-1. RUN NO. 10 RESULTS

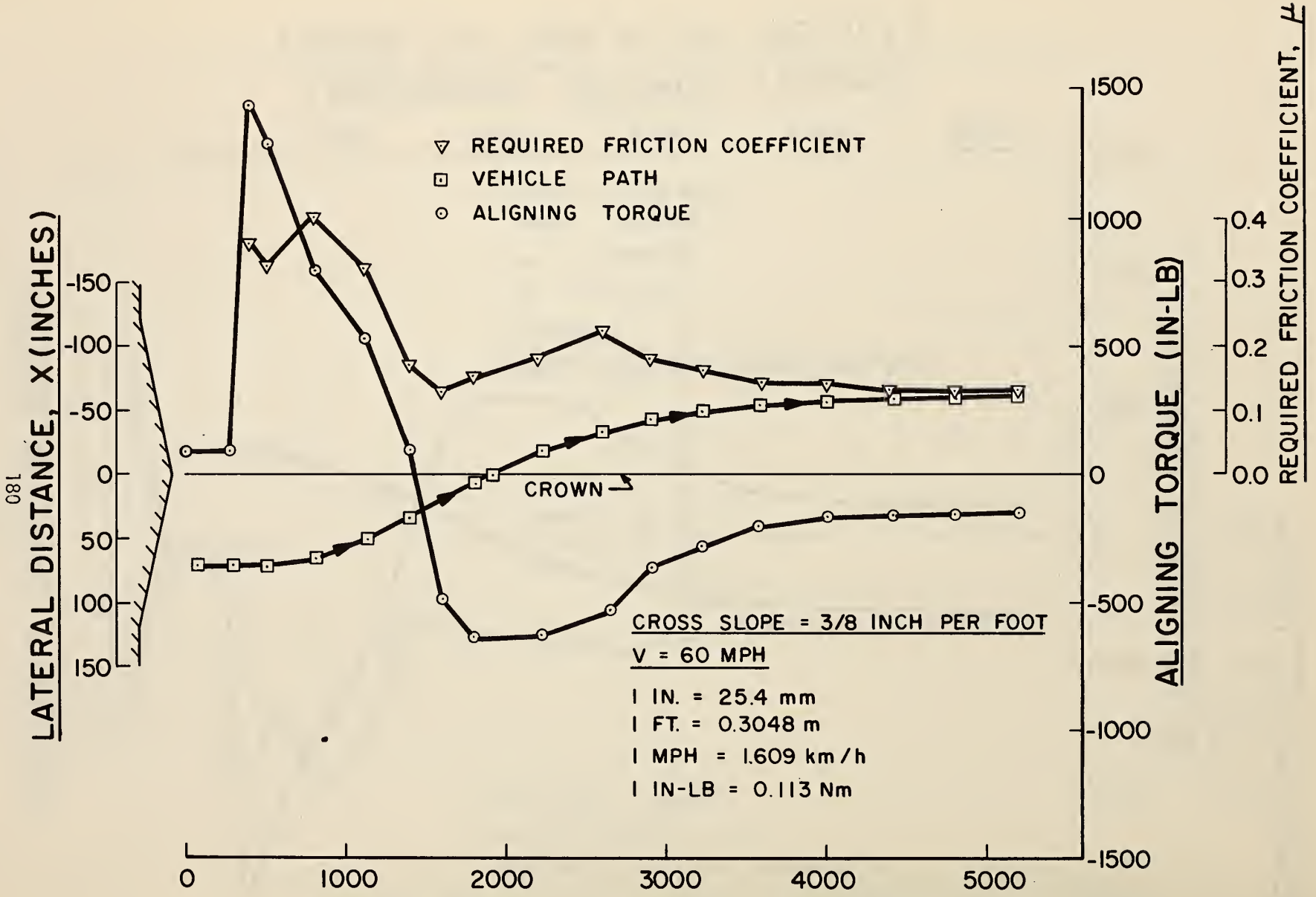


FIGURE B-2. RUN NO. 11 RESULTS

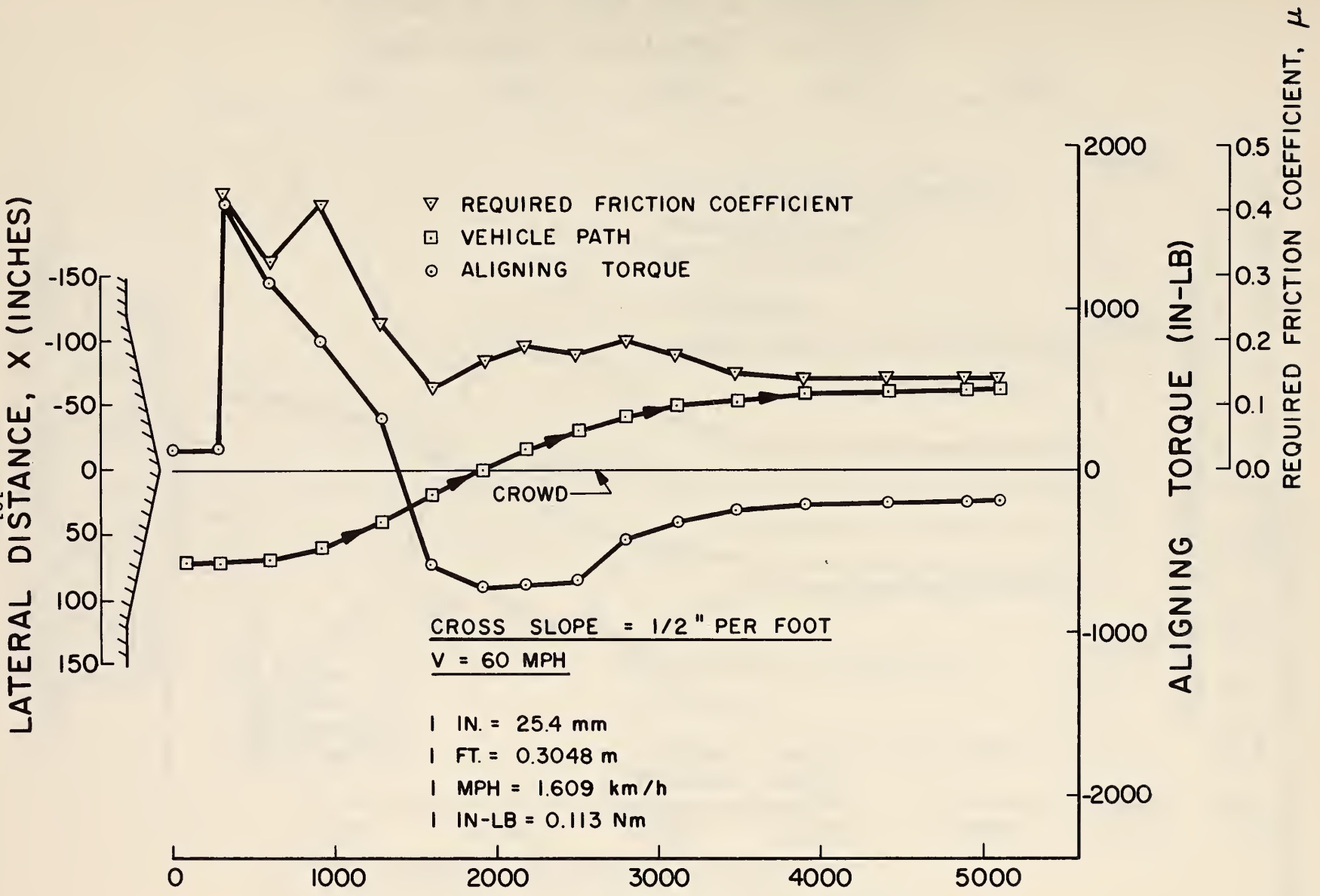


FIGURE B-3. RUN NO. 12 RESULTS

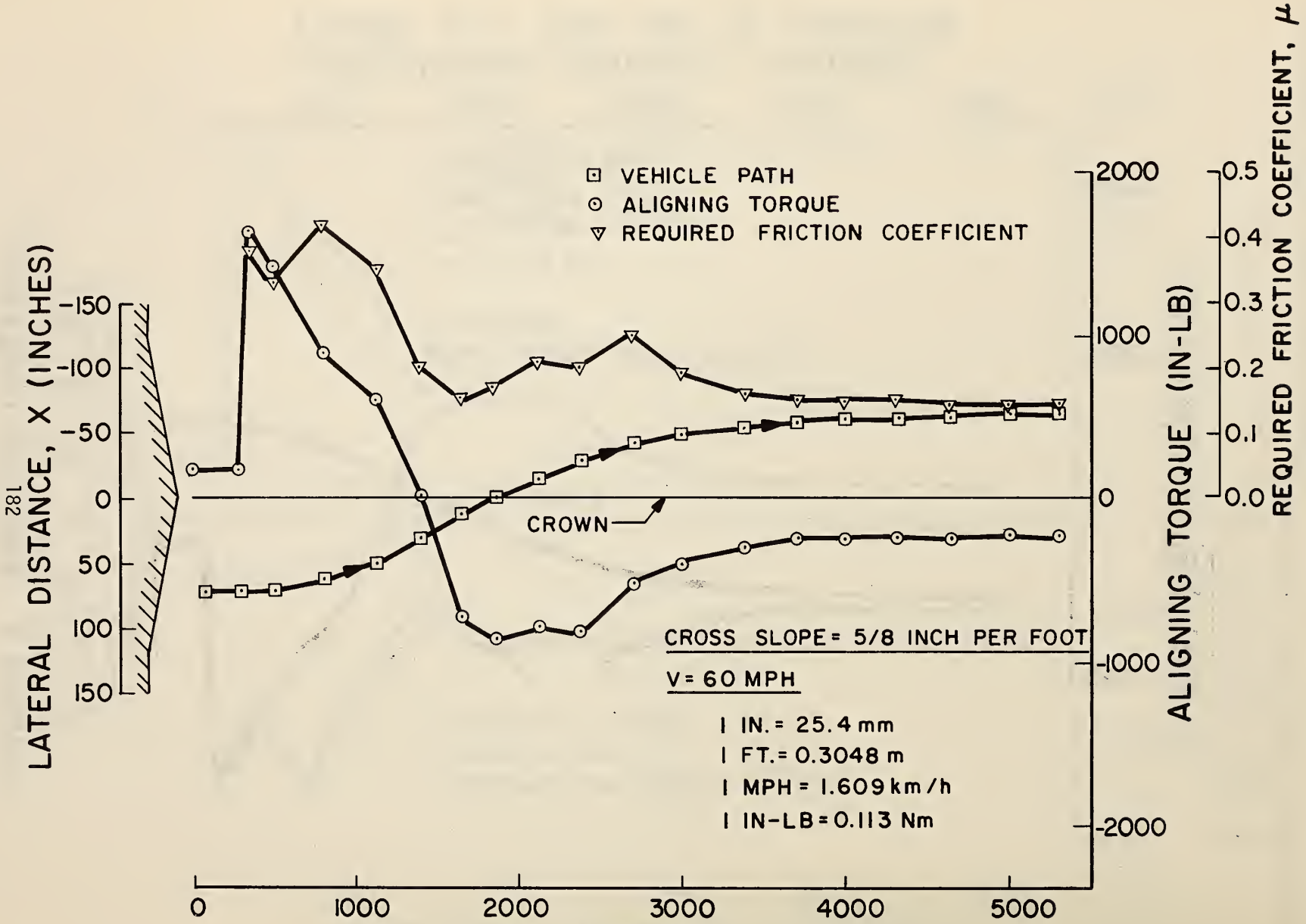


FIGURE B-4. RUN NO. 13 RESULTS

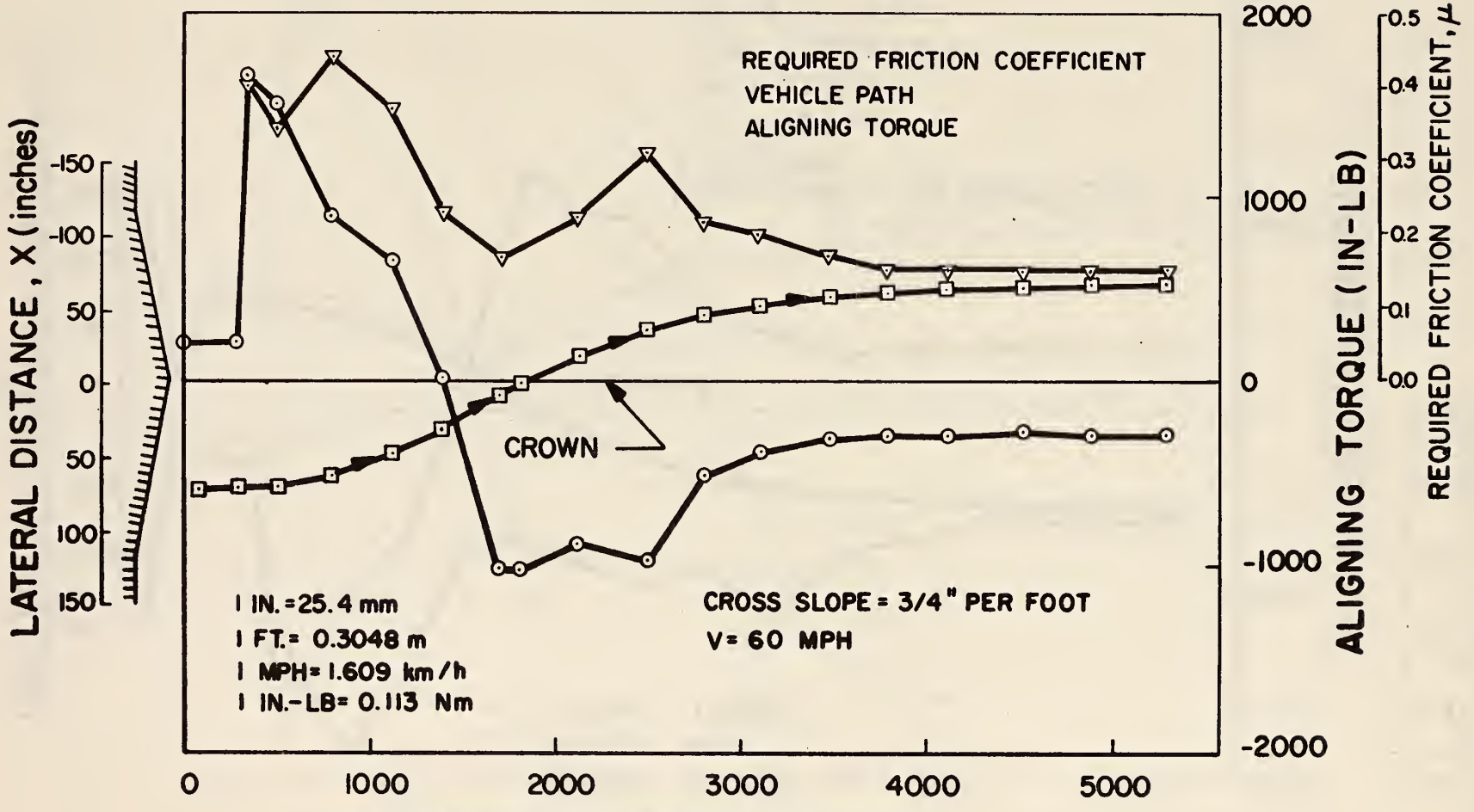


FIGURE B-5. RUN NO. 14 RESULTS

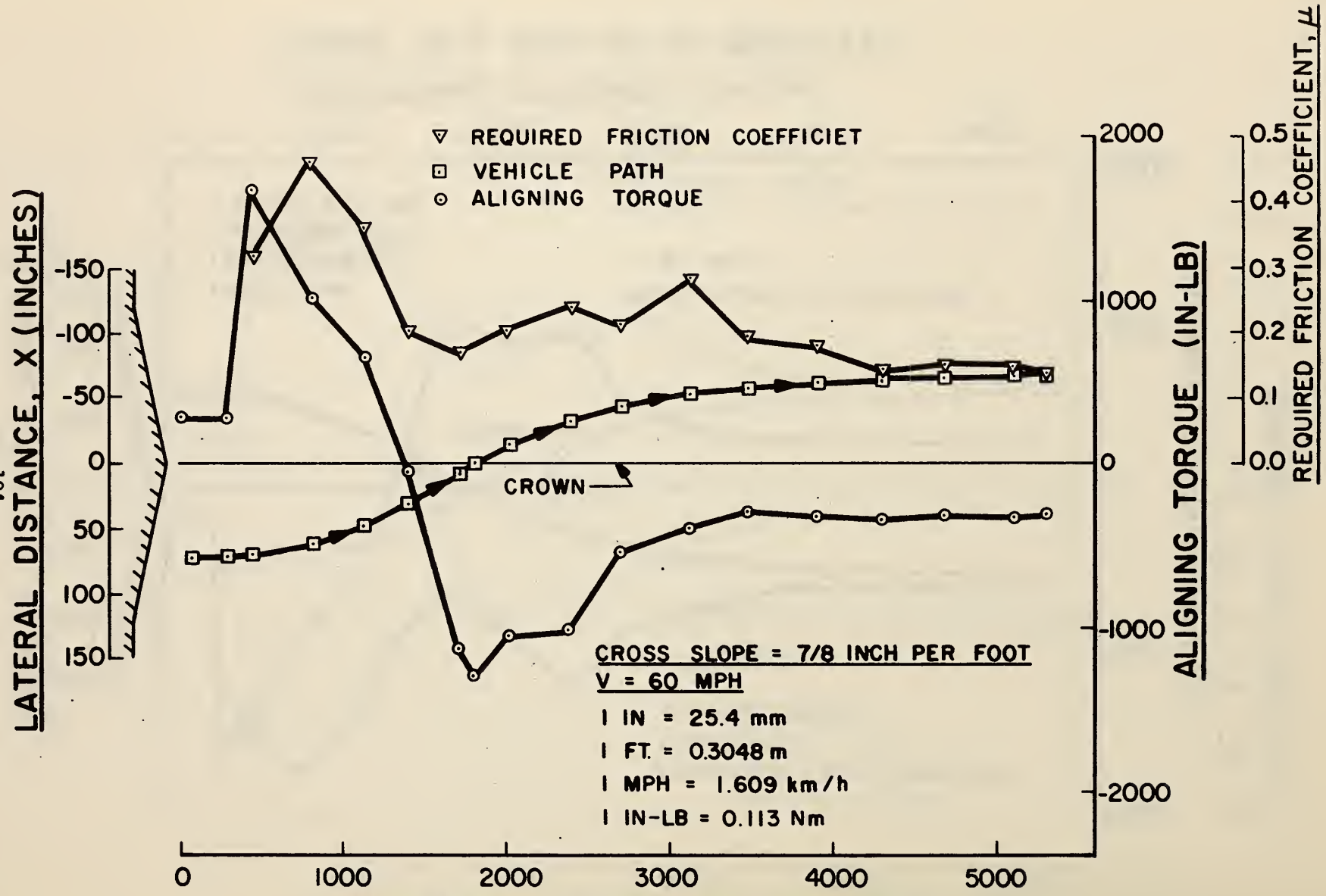


FIGURE B-6. RUN NO. 15 RESULTS

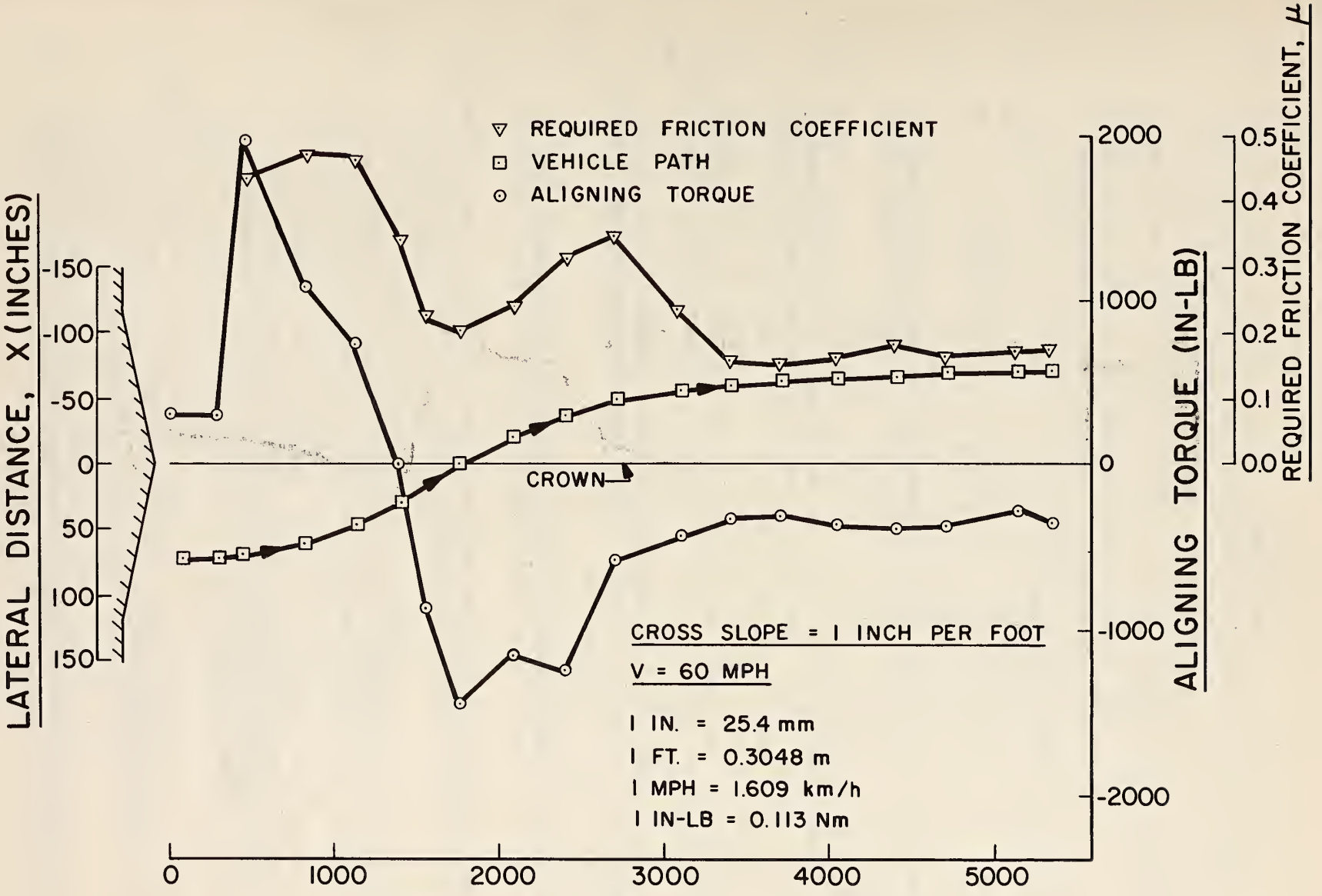


FIGURE B-7. RUN NO. 16 RESULTS

REFERENCES

1. Browne, Alan L., "A Mathematical Analysis for Pneumatic Tire Hydroplaning", Paper presented at ASTM winter meeting, Bal Harbour, Florida, Dec. 6, 1973.

2. Yeager, R.W., "Tire Hydroplaning: Testing, Analysis and Design", Physics of Tire Traction, edited by D.F. Hays and A.L. Browne, Plenum Press, 1974, pp.25-63.

3. Horne, W.B. and Dreher, R.C., "Phenomena of Pneumatic Tire Hydroplaning", NASA Technical Note TN D-2056, Nov. 1963.

4. Horne, W.B. and Joyner, U.T., "Pneumatic Tire Hydroplaning and Some Effects on Vehicle Performance", SRE International Automotive Engineering Congress, Detroit, Michigan, Jan. 1965.

5. Gengenbach, Werner, "The Effect of Wet Pavement on the Performance of Automobile Tires", Universität Karlsruhe, Deutschland (translated for limited distribution by CALSPAN, Buffalo, New York, July 1967).

6. Kummer, H.W. and Meyer, W.E., "Tentative Skid-Resistance Requirements for Main Rural Highways", NCHRP Report 37, Highway Research Board, 1967.

7. Gallaway, B.M., et al, "Influence of Water Depths on Friction Properties of Various Pavement Types", Research Report 138-6 , Texas Transportation Institute, Apr. 1973.

8. Ackers, P., "Resistance of Fluids Flowing in Channels and Pipes", Hydraulics Research Paper No.1, (HMSO), London.

9. Mayer, Paul G., "Roll Waves and Slug Flows in Inclined Open Channels", ASCE Proceedings H47, July 1959.

10. Lighthill, M.T. and Whitman, G.B., "On Kinematic Waves. I. Flood Movement in Long Rivers", Royal Society of London, Proceedings, 2794, pp. 281-316, May 1955.

11. Gallaway, B.M., Schiller, R.E.,Jr. and Rose, J.G., "The Effects of Rainfall Intensity, Pavement Cross Slope, Surface Texture and Drainage Length on Pavement Water Depths", Texas Transportation Institute Report 138-5, 1971.

12. Ross, N.F. and Russam, K., "The Depth of Rain Water on Road Surfaces", Ministry of Transport, Road Research Laboratory, Report LR-236, Great Britain, 1968.

13. _____, American Association of State Highway Officials, A Policy on Geometric Design of Rural Highways (AASHO "Blue Book"), Washington, D.C., 1965.

14. Rizenbergs, R.L., Burchett, J.L. and Napier, C.T., "Accidents on Rural Interstate and Parkway Roads and Their Relation to Pavement Friction", Kentucky Department of Transportation, Frankfort, Kentucky, Oct. 1973.

15. Holmes, T., Lees, G. and Williams, A.R., "A Combined Approach to the Optimization of Tyre and Pavement Interaction", Wear, Vol. 20, pp. 241-276, 1972.

16. Sabey, B.E., "Road Surface Characteristics and Skidding Resistance", Journal British Granite and Whinstone Federation, Vol. 5, No. 2, pp. 31-42, 1965.

17. Britton, S.C., "Standard Reference Pavement Surfaces for the Evaluation of Skid Test Equipment", Ph.D. Dissertation, Texas A&M University, Civil Engineering Department, Dec. 1972.

18. McCullough, B.F. and Hankins, K.D., "Skid Resistance Guidelines for Surface Improvements on Texas Highways", Highway Research Board, Highway Research Record No. 131, pp. 204-217, 1966.

19. Mahone, D.C. and Runkle, S.N., "Pavement Friction Needs", Highway Research Board, Highway Research Record No. 396, pp. 1-20, 1972.

20. Hayes, G.G. and Ivey, D.L., "Texture Skid Resistance and the Stability of Automobiles in Limit Maneuvers", Presented at the 1974 Annual Meeting of ASTM, June 1974.

21. Ivey, D.L., Keese, C.J., Neill, A.H.,Jr. and Brenner, F.C., "Interaction of Vehicles and Road Surfaces", Highway Research Board, Highway Research Record No. 376, pp. 40-53, 1971.

22. Bajer, J.J., "Proposal for a Procedure for Evaluating Wet Skid Resistance of a Road-Tire-Vehicle System", Society of Automotive Engineers Paper 690526, May 1970.

23. Bond, R., Katekhda, I.E.D., Lees, G. and Williams, A.R., "Road Surface Texture Characteristics and Tyre Performance", Presented at the 77th Annual ASTM Meeting, June 1974.

24. Veith, A.G. and Pottinger, M.G., "Tire Wet Track Operational Severity and Its Influence on Performance", The Physics of Tire Traction - Theory and Experiment, Plenum Press, New York, pp. 5-19, 1974.

25. Bond, R., Lees, G. and Williams, A.R., "Effective Design of Pavement Texture", The Physics of Tire Traction - Theory and Experiment, Plenum Press, New York, pp. 339-360, 1974.

26. Balmer, G.G., "The Significance of Pavement Texture", Report FHWA-RD-75-12, Washington, D.C.

27. Rose, J.G., Gallaway, B.M. and Hankins, K.D., "Macrotexture Measurements and Related Skid Resistance at Speeds from 20 to 60 Miles per Hour", Highway Research Board, Highway Research Record No. 341, pp. 33-45, 1970.

28. Tomita, H., "Effects of Pavement Surface Characteristics on Skid Resistance", M.S. Thesis, Texas A&M University, Mar. 1971.

29. Rose, J.G., Hutchinson, J.W. and Gallaway, B.M., "A Summary and Analysis of the Attributes of Methods of Surface Texture Measurements", ASTM Committee E-17 Symposium on Skid Resistance, Annual Meeting in Los Angeles, June 1972.

30. Schonfield, R., "Skid Numbers from Stereo-Photographs", Highway Research Board, Highway Research Record No. 311, pp. 11-25, 1970.

31. Moore, D.F., "Prediction of Skid Resistance Gradient and Drainage Characteristics for Pavements", Highway Research Board, Highway Research Record No. 131, pp. 181-203, 1966.

32. Gallaway, B.M., Epps, J.A., "Plant Mix Seals - Mix Design and Pilot Tests", Texas Transportation Institute Research Report 36-1F, Oct. 1974.

33. Gallaway, B.M., Rose, J.G., Schiller, R.E.,Jr. and Scott, W.W., "Influence of Water Depth on Friction Properties of Various Pavement Types", Texas Transportation Institute Research Report 138-6, Oct. 1974.

34. Ray, Gordon K. and Norling, L.T., "More Microtexture in Concrete Pavement for Greater, Longer Lasting Skid Resistance", Portland Cement Association, Skokie, Illinois, 18 pp.

35. Zube, E., Skog, J.B. and Kemp, G.R., "Field and Laboratory Studies on Skid Resistance of Pavement Surfaces", California Division of Highways, HPR-1(5), B-3-1.

36. Fleming, E.M., "Coefficients of Friction Between Rubber Tires and Concrete Road Surfaces", Proceedings, Highway Research Board, Vol. 14, pp. 178-188, 1934.

37. Rose, J.G. and Ledbetter, W.B., "Summary of Surface Factors Influencing the Friction Properties of Concrete Pavements", Highway Research Record No. 357, NRC, pp.53-63, 1971.

38. Grunan, D.L. and Michael, H.C., "Skid Characteristics of Pavement Surfaces in Indiana", Highway Research Board Bulletin 139, 1956.

39. Murphy, W.E. and Maynard, D.P., "Skid-Resistant Textures on Concrete Pavements - Research and Practice in the United Kingdom", pending publication by the Transportation Division of ASCE.

40. Mullen, W.G., et al., "Surface Wear and Skid Resistance Properties for Portland Cement Concrete Pavements", Report No. ERSD-110-74-3, N.C. State University, Raleigh, North Carolina, May 1974, 60 pp.

41. _____, "An Investigation of Skid Resistance in Missouri", Missouri State Highway Department Study No. 69-3, Aug. 1973, 111 pp.

42. Ganung, George A., "Friction Characteristics of Paving Materials in Connecticut", Report 1 on Research Project HPR 356, State of Connecticut Department of Transportation, Bureau of Highways, Apr. 1973, 61 pp.

43. Ganung, George A. and Christman, Robert, "Development and Implementation of a Skid Test Program in Connecticut", Report 2 on Research Project HS 412-002-180, State of Connecticut Department of Transportation, Bureau of Highways, Apr. 1972, 52 pp.

44. Davis, J.L., Ledbetter, W.B. and Meyer, A.H., "First Progress Report on Concrete Experimental Test Sections in Brazos County, Texas", Research Report 141-2, Texas Transportation Institute, Aug. 1973, 63 pp.

45. Thornton, J.B., "Determination of Desireable Finishes for Concrete Pavements", Final Report, Project 71-10, Georgia Department of Transportation, 1974.

46. Committee A2F01, "Surface Wear of Portland Cement Concrete Pavements", Pending publication as a Circular by the Highway Research Board, NRC.

47. Sabey, B.E., et al., "Factors Affecting the Friction of Tires on Wet Roads", SAE Paper No. 700376, International Automotive Safety Conference Compendium, pp. 324-340, 1970.

48. Horne, Walter B., et al., "Recent Research on Ways to Improve Tire Traction on Water, Slush or Ice", DeIngenieur, (Netherlands), Jrg. 78, Nr. 39, pp. 39-50, Sept. 30, 1966.

49. Stocker, A.J. and Lewis, J.M., "Variables Associated with Automobile Tire Hydroplaning", Research Report No. 147-2, Texas Transportation Institute, Texas A&M University, Sept. 1972, 84 pp.

50. Ledbetter, W.B., Meyer, A.H. and Ballard, D.E., "Evaluation of Full Scale Experimental Concrete Highway Finishes", Research Report 141-4F, Texas Transportation Institute, Texas A&M University, in review.

51. Hocking, R.R. and Leslie, R.N., "Selection of the Best Subset in Regression", Technometrics, Vol. 9, pp. 531-540, 1967.

52. Lamotte, L.R. and Hocking, R.R., "Computational Efficiency in Selection of Regression Variables", Technometrics, Vol. 12, pp. 83-93, 1970.

53. Smith, Richard B. and Silva, Edward L., "Summary of Replies to Questionnaire in Inventory of Practices and Operational Experience as Related to the Design Features of Horizontal Alignment and Pavement Cross Slope", Unpublished report by the AASHO Operating Subcommittee on Roadway Design, Apr. 1972.

54. McHenry, R.R. and Segal, D.J., "Determination of Physical Criteria for Roadside Energy Conversion Systems", Cornell Aeronautical Laboratory Report No. VJ-2251-V-1, July 1967.

55. McHenry, R.R. and Deleys, N.J., "Vehicle Dynamics in Single Vehicle Accidents: Validation and Extension of a Computer Simulation", Cornell Aeronautical Laboratory Report No. VJ-2251-V-3, December 1968.

56. McHenry, R.R. and Deleys, N.J., "Automobile Dynamics - A Computer Simulation of Three-Dimensional Motions for Use in Studies of Braking Systems and of the Driving Task", Cornell Aeronautical Laboratory Report No. VJ-2251-V-7, Aug. 1970.

57. Dunlap, D.F., et al., "Influence of Combined Highway Grade and Horizontal Alignment on Skidding", NCHRP Project 1-14, Final report draft, Highway Safety Research Institute, Univ. of Michigan, Ann Arbor, Michigan, Dec. 1973.

58. Weaver, G.D., Hankins, K.D. and Ivey, D.L., "Factors Affecting Vehicle Skids: A Basis for Wet Weather Speed Zoning.", Texas Transportation Institute Research Report 135-2F, Feb. 1973.

59. Hurnall, D.D. and Noble, A.M., "Drainage of Wide Flat Pavements", Country Roads Board, Victoria, Australia, unpublished paper.

60. _____, "A Policy on Design of Urban Highways and Arterial Streets", American Association of State Highway Officials, 1973.

61. _____, "Street and Highway Drainage, Volume 2 - Design Charts", Institute of Transportation and Traffic Engineering, Univ. of California, 1965.

62. Hankins, Kenneth D., Morgan, Richard B., Ashkar, Bashar and Tutt, Paul R., "The Degree of Influence of Certain Factors Pertaining to the Vehicle and the Pavement on Traffic Accidents Under Wet Conditions", Texas Highway Department, Departmental Research, Report No. 133-3F, Sept. 1970.

63. Gallaway, Bob M. and Rose, Jerry G., "Macro-Texture, Friction, Cross Slope and Wheel Track Depression Measurements on 41 Typical Texas Highway Pavements", Research Report 138-2, Texas Transportation Institute, June 1970.

64. Ivey, D. L. and Lehtipuu, Eero, K., "Rainfall and Visibility - The View from Behind the Wheel", Research Report 135-3, Texas Transportation Institute, Aug. 1974

65. Weaver, G., et al., "Wet Weather Speed Zoning". Highway Research Record No. 471, 1973. Presented at the Jan. 1973 meeting of the Highway Research Board, Washington, D. C..

66. Ivey, D. L., et al.; "Side Friction Factors Used in the Design of Highway Curves". Presented at the Jan. 1973 meeting of HRB in Washington, D. C.. Available from NCHRP.

67. Van Til, C. J., Carr, B. J. and Vallerga, B.A., "Requirements for Wear-Resistant and Skid-Resistant Highway Pavement Surfaces", Interim Report NCHRP 1-12(3), Aug. 1973.

68. Smith, R.W., Rice, J.M. and Spelman, S.R., "Design of Open-Graded Asphalt Friction Courses", FHWA-RD-74-2, Interim Report, Jan. 1974.

69. Gallaway, B.M., and Epps, J.A., "Mixture Design Concepts, Laboratory Tests and Construction Guides for Open-Graded Bituminous Overlays", Research Report 36-1F, Texas Transportation Institute, Oct. 1974.

70. Brown, J.R., "An Experiment Comparing the Performance of Roadstones Used as Chippings in Rolled Asphalt", Ministry of Transport, Roads Research Laboratory, Report LR63, Great Britian, 1967.

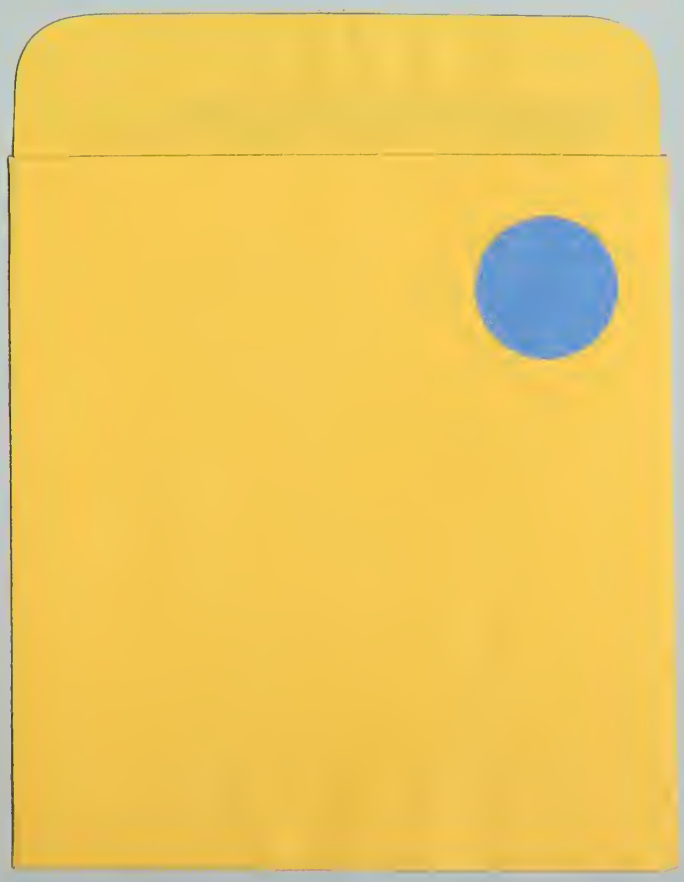
71. Tyler, R.L., "Sprinkle Treatment for Achieving Skid Resistant Pavements in Texas", Departmental Research Report No. SS 20.0, Texas Highway Department, Aug. 1974.

72. Maupin, G.W., "Sprinkle Treatment Increases Highway Skid Resistance", Civil Engineering, Feb. 1972.

73. Woods, Donald L., "The Driver and the Road," Proceedings of the American Society of Civil Engineering, Transportation Engineering Journal, TE4, Nov. 1971.

74. Woods, Donald L., "Application of Driver Expectancy in Design," Proceedings of the Kansas Highway Engineering Conference, Nov. 1972.

U.S. GOVERNMENT PRINTING OFFICE: 1976 [illegible] 73/399



DOT LIBRARY
00179809